DRAFT GEOTECHNICAL REPORT
EDGEWATER CREEK BRIDGE REPLACEMENT
EVERETT, WASHINGTON

HWA Project No. 2019-157-21

July 30, 2021

Prepared for:

Trantech Engineering

HWA GeoSciences Inc.

July 30, 2021
HWA Project No. 2019-157-21

Trantech Engineering
365 118th Ave. SE Suite 100
Bellevue, WA 98005

Attention: Khashayar Nikzad, PhD, P.E.

Subject: **DRAFT GEOTECHNICAL REPORT**
                Edgewater Creek Bridge Replacement
                Everett, Washington

Dear Mr. Nikzad:

We are pleased to present this updated draft geotechnical report prepared in support of the proposed replacement of Edgewater Creek Bridge in Everett, Washington. This report supersedes all previous versions. The purpose of this study was to evaluate the soil and ground water conditions around the existing bridge and provide geotechnical recommendations in support of the proposed replacement. Several design aspects of the project have not been determined to date. Therefore, we expect this report to be revised as the project progresses.

We appreciate the opportunity to provide geotechnical engineering services on this project. If you have any questions regarding this report or require additional information or services, please contact us at your convenience.

Sincerely,

HWA GEOSCIENCES INC.

Sean Schlitt, P.E.   Donald Huling, P.E.

Geotechnical Engineer   Geotechnical Engineer, Principal

TABLE OF CONTENTS

List of Figures (Following Text)

Appendices

Appendix A: Field Exploration

Appendix B: Laboratory Program

Appendix C: Geologic Profile C-C' (Bridge Alignment) Global Slope Stability Analysis

Appendix D: Slope Stability Profile A-A' (North Slope) Global Slope Stability Analysis

Appendix E: Slope Stability Profile B-B' (Critical Slope) Global Slope Stability Analysis

Appendix F: LPILE Parameters

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1.0 INTRODUCTION

1.1 GENERAL

This report presents the results of a geotechnical engineering study performed by HWA
GeoSciences Inc. (HWA) in support of Edgewater Bridge Replacement project in Everett,
Washington. The purpose of this study was to evaluate the soil and ground water conditions
along the alignment to aid in a bridge replacement project.

Our work for this project included performing a site reconnaissance, preparing and conducting a
site investigation program, performing geotechnical engineering analyses, and providing
recommendations for geotechnical aspects of design. Their field work included drilling six (6)
machine-drilled borings in support of bridge foundation design, three (3) machine-drilled borings
in support of the wing wall design, four machine drilled borings in support of critical slope
stability evaluations, two (2) machine-drilled borings in support of the LaMar Drive soldier pile
wall, and four (4) shallow hand borings to evaluate slope conditions. Additionally, HWA
conducted a series of slope reconnaissances to evaluate the stability of the slope and verify the
geometry of several critical slope features.

Appropriate laboratory tests were conducted on selected soil samples from each of our
exploration phases to determine relevant engineering properties of the subsurface soils. In this
report, we present a summary of the subsurface and ground water conditions observed, as well as
design and construction recommendations for the bridge replacement.

1.2 PROJECT DESCRIPTION

It is our understanding that the City of Everett (City) would like to replace the Edgewater Creek
Bridge along Mukilteo Boulevard in Everett, Washington. The approximate location of the
project corridor is shown on the Vicinity Map, Figure 1. The Edgewater Bridge was constructed
in 1946 and is founded on concrete pilings. Previous analysis and bridge inspections indicated
that the bridge has become deficient and seismically vulnerable. The proposed project will
completely remove and replace the existing bridge structure with a new structure.

2. FIELD AND LABORATORY TESTING

2.1 GEOTECHNICAL FIELD INVESTIGATION

Our geotechnical exploration program included surface reconnaissance of the alignment, drilling
Thirteen (13) machine-drilled borings and completion of four (4) shallow handhole geotechnical

borings over the course of six phases of work, as described below. The approximate locations of these borings are shown on the Site and Exploration plan, Figure 2. Logs for each boring through each phase are presented in Appendix A of this report.

Phase 1: Phase 1 of our field exploration program consisted of drilling four borings, designated BH-1 through BH-4. Borings BH-1 and BH-4 were drilled within the abutment portion of the bridge structures to a depth of approximately 101.5 feet below ground surface (bgs) in support of design of the proposed east and west abutments (Pier 1 and Pier 4). Borings BH-2 and BH-3 were attempted through the bridge deck in support of preliminary bridge design analysis and design of the proposed interior piers. Due to challenges associated with containing drilling fluid, these borings were terminated at depths ranging from approximately 36.5 to 41 feet bgs and an alternative drilling method was determined to be required and were re-drilled in Phase 3. Phase 1 of drilling was performed between March 16th-20th, 2020 by Holocene Drilling of Puyallup, Washington, under subcontract to HWA using a truck-mounted Dietrich D-120 drill rig using mud rotary drilling method.

Phase 2: Phase 2 of our field exploration program consisted of drilling two borings, designated BH-6 and BH-7. Both borings were drilled on the slope below Mukilteo Lane to a depth of approximately 14 and 15 feet bgs, respectively, in support of determination of the stability of the existing slope. Phase 2 was completed on March 30th, 2020 by Geologic Drill Partners of Bellevue, Washington, under subcontract to HWA, using a limited access Acker drill equipped with hollow stem augers.

Phase 3: Phase 3 of our field exploration program consisted of advancing two new borings near the locations of previously completed borings BH-2 and BH-3 that were terminated early (new borings designated as BH-2A and BH-3A). BH-2A and BH-3A were advanced to depths of about 76.5 feet bgs. Additionally, boring BH-5 was completed along Mukilteo Lane to a depth of approximately 31.5 feet below ground surface in support of design of the abutment wing walls. It should be noted that borings BH-2A and BH-3A were completed in the opposite travel lanes from the previous attempt; therefore, differences in surface elevations were noted. These borings were performed between March 31st – April 3rd, 2020 by Holocene Drilling of Puyallup, Washington, under subcontract to HWA using a track-mounted GeoProbe 8140LC employing Sonic drilling.

Phase 4: Phase 4 of our field exploration program consisted of digging four handholes, designated HH-1 through HH-4. Each of these handholes were excavated and drilled below the existing bridge structure alignment, within the ravine to depths ranging from approximately 7 to 7.7 feet bgs in support of investigation of subsurface soil and ground water conditions within the ravine. Phase 4 was completed in March 5th, March 30th, and April 3rd, 2020 by HWA geologists using hand operated drilling and sampling equipment.

Phase 5: Phase 5 of our field exploration program consisted of drilling 4 borings, designated BH-8 through BH-11. Each boring was drilled on the ravine slope below W Mukilteo Boulevard to a maximum depth of approximately 16.5 feet bgs, in support of determination of the stability of the existing slope. The locations of the borings were planned to better map the critical ravine slope and the observed failure scarp; two borings were placed at the base of this scarp (BH-8 and BH-9) and two at the top of the scarp (BH-10 and BH-11). Phase 5 was completed on September 9th and 10th, 2020 by Geologic Drill Partners of Bellevue, Washington, under subcontract to HWA, using a limited access Acker drill equipped with hollow stem augers.

Phase 6: Phase 6 of our field exploration program consisted of drilling 2 borings, designated BH-12 and BH-13. Each boring was drilled within the city right-of-way to the southeast of LaMar Drive, to maximum depths of approximately 51.5 and 46.5 feet bgs respectively. The borings were drilled to determine soil engineering characteristics along the alignment of the proposed soldier pile wall along LaMar Drive. Phase 6 was completed on July 16, 2021 by Holocene Drilling of Puyallup, Washington, under subcontract to HWA, using a Dietrich D-50 track rig using hollow stem auger.

Standard Penetration Testing (SPT) was performed in each boring for phases 1, 2, and 3 using a 2-inch outside diameter, split-spoon sampler driven by a 140-pound automatic hammer. Standard Penetration Testing (SPT) was performed in borings for phase 5 using a 2-inch outside diameter, split-spoon sampler driven by a 140-pound cat head pulley hammer. During the test, a sample was obtained by driving the sampler 18 inches into the soil with the hammer free-falling 30 inches. The number of blows required for each 6 inches of sampler penetration was recorded. The N-value (or resistance in terms of blows per foot) is defined as the number of blows recorded to drive the sampler the final 12 inches. If a total of 50 blows was recorded within a single 6-inch interval, the test was terminated, and the blow count was recorded as 50 blows for the number of inches of penetration achieved. This resistance, or N-value, provides an indication of the relative density of granular soils and the relative consistency of cohesive soils.

Additionally, a larger 3-inch outside diameter, Dames & Moore sampler was utilized at specific depths during phase 3 in order to collect ring samplers to conduct direct shear tests. The samples collected with this sampler (BH-2A, S-2 and S-9; BH-3A, S-3 and S-9) have blow counts that do not reflect standardized values as they utilized the larger Dames & Moore sampler with the standard 140-lb hammer. These values have been adjusted in our analyses to reflect standard SPT N-value blow counts for the purpose of our design. The machine-drilled boring logs are presented on Figures A-2 through A-30.

For the handholes conducted in phase 4, Dynamic Cone Penetrometer (DCP) testing was performed at each boring location to assess subsurface soil and groundwater conditions. The DCP equipment consists of a steel extension shaft assembly, with a 60-degree hardened steel cone tip attached to one end, which is driven into the soil by means of a sliding drop hammer.

The base diameter of the cone is 20 mm (0.79 inches). The diameter of the shaft is 8 mm (0.315 inches) less than the cone, to reduce rod friction at shallow penetration depths. The DCP is driven by repeatedly dropping an 8-kg (17.6-pound) sliding hammer from a fixed height of 575 mm (22.6 inches). The depth of cone penetration is measured after each hammer drop and the in-situ shear strength of the soil is reported in terms of the DCP index. The index is based on the average penetration depth resulting from 1 blow of the 8-kg (17.6-pound) hammer and is reported as millimeters per blow (mm/blow). The data obtained from the DCP tests was then correlated to Standard Penetration Test (SPT) N-value (blows/foot), to evaluate the strength of the subgrade soils. The DCP data, converted to SPT N-value (blows/foot), is plotted on Figures A-31 through A-34 for the hand auger borings.

Samples in phases 1, 2, and 5 were obtained in the borings at approximately 2.5-foot intervals to a depth of 20 feet bgs then at approximately 5-foot intervals to the bottom of the boring. Samples in phase 6 were obtained in the borings at approximately 2.5-foot intervals to a depth of 15 feet bgs then at approximately 5-foot intervals to the bottom of the boring.

Given the continuous sampling nature of Sonic Drilling and handhole borings, samples in phases 3 and 4 were collected when changes were observed as well as on a 5-foot interval. At certain depths in the sonic borings, sonic-induced liquefaction of saturated sands resulted in the drill casing sinking below the bridge deck, such that a 5-foot interval SPT sample could not be obtained. After termination, boreholes greater in depth than 10 feet were abandoned and backfilled with bentonite chips per Department of Ecology requirements. The shallow handholes were abandoned with drilling cuttings and native soil.

The explorations were completed under the full-time observation of a geotechnical engineer or engineering geologist from HWA, who collected pertinent information including soil sample depths, stratigraphy, soil engineering characteristics, and ground water occurrence as the exploration was advanced. Soils were classified in general accordance with the classification system described on Figure A-1, which also provides a key to the exploration log symbols. The exploration logs are presented on Figures A-2 through A-34.

The stratigraphic contacts shown on the individual logs represent the approximate boundaries between soil types. Actual transitions may be more gradual. The soil and ground water conditions depicted are only for the specific dates and locations reported, and therefore, are not necessarily representative of other locations and times.

2.2 LABORATORY TESTING

Laboratory tests were conducted on selected samples retrieved from the explorations to characterize relevant engineering properties and index parameters of the soils encountered at the site. The tests included visual classification, natural moisture content determination, organic content, Atterberg Limits, direct shear analysis, and grain size distribution analysis. The tests

were conducted in the HWA laboratory in general accordance with appropriate American Society of Testing and Materials (ASTM) standards and are discussed in further detail in Appendix B. The test results are also presented in Appendix B, and/or displayed on the exploration logs in Appendix A, as appropriate.

3. SITE CONDITIONS

3.1   GENERAL

The existing bridge is constructed along W Mukilteo Boulevard near the western city limits of Everett, Washington. The existing bridge structure was constructed in 1946 and is founded on concrete pilings. The prospect site is located within a residential neighborhood that is positioned atop a bluff that overlooks Puget Sound. Previous analysis and bridge inspections indicated that the Edgewater Bridge has become deficient and seismically vulnerable. From the visual observation, the deck of the bridge has experienced distress, with transverse and longitudinal cracks. The bridge crosses a steep ravine that is approximately 85-feet at the deepest point.

3.2   GENERAL GEOLOGIC CONDITIONS

Specific geologic information for the project area was obtained from the *Geologic map of the Everett 7.5-minute quadrangle, Snohomish County* (Minard, 1986). According to this map, near-surface deposits beneath the bridge alignment consist of Transitional Beds over the Whidbey Formation. The Transitional Beds consist of clay, silt, and very fine sand deposited in still to slowly moving water. These were deposited in lake environments during the transition from non-glacial to glacial conditions in the Puget Lowland at the beginning of the latest continental glaciation. Glacial ice known as the Puget Lobe of the Cordilleran Ice Sheet advanced southward from present British Columbia, depositing glacial "flour" (silt and clay) from meltwater in lakes ahead of the ice. Beneath this unit, non-glacial river-deposited and marine-deposited, stratified sand and silt are present. This material is known as the Whidbey Formation, and contains terrestrial and marine organic matter which indicate a climatic regime similar to the present.

Subsequent to deposition of the Transitional Beds, the deposits were covered by Advance Outwash sand deposited by meltwater directly in front of the ice, then by Till formed beneath the ice as the glacier rode over the deposits. As a result, the deposits beneath the glacier including the Transitional Beds and Whidbey Formation were over-consolidated to a hard consistency or very dense condition by the weight of over 3,000 feet of ice.

3.3   SUBSURFACE CONDITIONS

The results of our subsurface explorations indicate that the bridge alignment is underlain by glacially over-consolidated Transitional Beds and Whidbey Formation Colluvial soils derived

from weathering, creep, and slope failure were encountered to depths of up to 17 feet along the slope. Our interpretation of the geologic conditions along the centerline of the proposed bridge are shown in Figure 3, Geologic Cross Section A-A'. An additional two (2) slope stability profiles have been produced to model the potential for slope failures: Slope stability Profile B-B' to the north of the wing walls and Profile C-C' along the critical slope. The alignment of the profiles is shown on the Site and Exploration Plan, Figure 2. The soil units encountered are described in more detail as follows:

- **Fill:** Fill was observed in borings BH-1, BH-4, and BH-5, BH-12, and BH-13 from the surface beneath the pavement or grass to depths of about 3 to 17.5 feet bgs. Fill soils encountered were typically very loose to dense, brown, silty sands and gravels with scattered organics and rootlets. Some fine-grained fill material was encountered at depth in boring BH-1, BH-12, and BH-13. In general, this fill material was inconsistent and may vary significantly across the site.

- **Colluvium:** Colluvium was observed in all borings and handholes except BH-1, BH-12, and BH-13, beneath the fill soils to the west and along the ravine slopes. These deposits ranged in thicknesses from about 2.5 to 17 feet and consisted of very soft to soft, olive gray to yellow-brown, silt with varying amounts of gravel and abundant organics. The colluvium material appeared to consist of multiple generations of deposited material including material deposited before the development of the existing bridge, as well as recently deposited material due to near-surface failures.

- **Transitional Beds:** Transitional beds were encountered in all borings and most handholes, extending from the base of the fill or colluvium soil. In explorations that extended through the full thickness of the unit, these deposits ranged in thicknesses from 13 to 62.5 feet and consisted of medium stiff to hard, olive-gray to brown, inelastic silts, lean clays, fat clays, and interspersed slightly silty and silty sands. Transitional beds consist of a combination of glaciolacustrine deposits and non-glacial lake deposits that alternate due to changing depositional environments.

- **Whidbey Formation:** Whidbey formation deposits were encountered in each of the six deep borings, BH-1, BH-2A, BH-3A, BH-4, BH-12, and BH-13. These deposits were encountered beneath the transitional bed unit and extended to the termination depth of each boring. The deposits predominantly consisted of very dense silty sands and gravels with the presence of trace marine shell fragments. A sequence of hard, dark gray, silt was also encountered at the bottom of BH-2A.

  Lower blow counts, observed in this unit, in borings BH-2A and BH-3A can likely be attributed to the vibratory method of Sonic Drilling used and are unrepresentative of the highly consolidated material. The Whidbey Formation consists of a combination of non-

glacial fluvial and lacustrine deposits, resulting in interbedding of sands and silts with trace marine remnants.

3.4 GROUND WATER

Ground water seepage was observed in borings on the slope. The depth to ground water was about 8.7 feet bgs in BH-2, 9.5 feet bgs in BH-2A, 7 feet bgs in BH-3A, 11 feet bgs in BH-3, 5.3 feet bgs in BH-8, and 1.9 feet bgs in BH-9. Ground water was not observed in borings BH-1, BH-5 through BH-7, BH-10, and BH-11 through BH-13. Handhole HH-4 encountered groundwater on the southeast slope at approximately 4 feet bgs. In BH-1 and BH-4, at the abutments, the ground water level was not observed due to the mud rotary drilling methods used. Ground water seepage was observed along the lower 10- to 15-feet of the ravine slopes, adjacent to the bridge. The most prominent seeps were on the southeastern slope, with multiple springs flowing overland to Edgewater Creek.

Ground water was encountered in boring BH-9 at a depth of approximately 10.75 feet bgs within the underlying sand soils. The overlying surficial fine-grained colluvium was observed to act as a discrete confining layer. Upon completion of this boring, groundwater was remeasured at a depth of approximately 1.9 feet bgs. The presence of rebounding groundwater conditions and groundwater springs suggests that artesian groundwater conditions may be encountered during drilled shaft excavations. Perspective contractors should be prepared to account for these artesian conditions. No artesian groundwater conditions were encountered in any of the other borings.

Based on ground water conditions observed in the boreholes and on the slope, it appears that the presence of ground water can be attributed to two sources: 1) perched water resting atop the relatively impermeable clay soils encountered in the transitional bed materials and 2) static water contained within the permeable sand soils capped by overlying impermeable clay soils producing artesian ground water conditions. The potential for the development of shallower ground water conditions during construction should be considered. Artesian conditions may potentially be encountered in locations beyond those observed in our explorations and should be anticipated during construction. We expect that the ground water will vary seasonally with the highest potential levels in the wet winter months and the lowest levels in the dry summer months. Additionally, water may be encountered emanating from saturated sand seams contained within the relatively impermeable transitional bed material.

The presence of ground water contributes greatly to instability of the lower slope, resulting in oversteepening and slope failure, and subsequent instability of the middle and upper slope. Running sand conditions should be expected in drilled shaft excavations.

3.5 MATERIAL PROPERTIES

The engineering material properties for the soil units underlying the proposed Edgewater Bridge Replacement were chosen based on a combination of laboratory test results, SPT correlations, and engineering judgement. The proposed soil properties are presented in Table 1.

Table 1.
Material Properties for the Soil Units in the Project Vicinity

Unit Name	γ (pcf)	c'(psf)	φ' (degrees)
Proposed Structural Fill	135	0	36
Existing Fill	120	0	34
Colluvium (Fine-Grained)	105	475	0
Transitional Beds (Fine-Grained)	110	1,500 – 4,500	0
Whidbey Formation (Coarse-Grained)	135	0	38

The material properties for the Transitional Beds, presented in Table 1, were calculated using laboratory test results and Mohr-Coulomb shear strength relationships. These deposits exhibited a combination of cohesive and cohesionless properties. Due to the limitations associated with the global slope stability analysis, slightly different, but equivalent, cohesive Transitional Bed material properties were calculated, using Mohr-Coulomb shear strength relationships, for use with slope stability models.

3.6 SLOPE RECONNAISSANCE

3.6.1 General

Slope conditions were observed in February, March, and July of 2020 by an HWA senior engineering geologist. The ravine slopes were traversed on foot and surface observations were made regarding topography, geomorphology, vegetation patterns and conditions, soil exposures, and ground water seeps.

The bridge crosses perpendicularly to the Edgewater Creek ravine, which trends northward with the stream flowing into Puget Sound. The ravine dissects gently rolling terrain which slopes overall northward, with steep bluffs adjacent to the sound. The stream channel is approximately 85 feet below the bridge deck surface, according to the site survey. The stream has a step-pool morphology, with large woody debris and rocks defining the steps. Springs were observed along the lower 10 to 15 feet of the shallow sloping area on either side of the stream. The ground

along the steep slopes was probed at intervals with a 3-foot long, ½-inch diameter, steel T-handled probe. The slope surface probed generally from 0.5 to 2.5 feet, with shallow probing at areas of soil exposure. The lower shallow sloping areas, adjacent to the stream, probed the full 3 feet of the probe in loose, wet, soils.

The site survey contours are shown on the Site and Exploration Plan, Figure 2, and Geologic Cross Section, Figure 3. Slope observations are described below by area relative to the bridge.

3.6.2 **Western Slope**

The western ravine slope, immediately beneath the bridge, is partly devoid of vegetation, especially in the upper half, likely due to lack of water. The ground generally slopes steeply to the northeast, with a shallow-sloping toe in the lower approximately 10 feet of elevation. An old concrete abutment is present under the bridge, approximately 10 to 20 feet from the present bridge abutment. The old abutment has square rebar protruding from the concrete. The historic abutment structure is not parallel to the existing bridge abutment, nor level at the top, which are indications that it has translated likely due to slope movement. It is likely that slope movement precipitated replacement of the older bridge. Soil exposures near the top of the slope consisted of finely bedded clay and silt with sand seams, typical of the Transitional Beds geologic unit described in previous sections of this report. The bedding dipped steeply downslope with no consistent strike direction, which indicate deformation due to slope movement.

The northwest slope (north of the bridge, west side) slopes steeply to the northeast from the bridge abutment wing wall and Mukilteo Lane, which trends northwest from Mukilteo Blvd. The upper 15- to 20-foot height of the slope, close to the bridge, is steeper than immediately below and is inclined at approximately 1H:1V (Horizontal:Vertical). Northwestward of the wing wall, a chronic slide area is evident from cracks and downsets of an old asphalt-paved walkway and distress within the margins of Mukilteo Lane. Rotated blocks of soil approximately 5 to 10 feet lower than the roadway and approximately 30 feet wide were evident to at least 340 feet along Mukilteo Lane from the wing wall. Vegetation in this area consisted of blackberry vines, alder trees (some with pistol-grip butts), some Bigleaf maple trees (topped for the view), and native brush. Surficial soils where exposed consisted of topsoil, in this case dark brown sandy silt.

The middle portion of the northwest slope was inclined at approximately 1.5H:1V and was gently hummocky, with local relief on the order of 3 to 5 feet and as steep as 0.5H:1V. Vegetation was predominantly sword fern and mature alder trees. Limited soil exposures at the steepest portions consisted of oxidized to gray silt and clay. The hummocky local relief appears to be a function of shallow slope failures and creep.

The lower 10 to 25 feet of the steep slope was inclined at approximately 0.5H:1V and relatively planar, indicative of a slide scarp complex, and extended from beneath the bridge to the

northwest and north, roughly parallel to the stream. This area was sparsely vegetated with sword ferns, and otherwise soil was exposed consisting of hard, gray clay and silt. The toe of this portion of the slope was approximately 10 feet higher than the stream.

The remainder of the northwest slope consisted of a very shallow apron extending to the stream, inclined at approximately 5H:1V and less. The ground surface was generally saturated, despite dry weather during times of reconnaissance (including during late July, 2020), with spots of groundwater seepage.

The southwest slope was somewhat similar in overall morphology to the northwest slope. However, it slopes generally east-northeast. Evidence of recent and old sliding was apparent, based on tree size and type, and soil exposures and topography. A young slide area is evident starting at the top of the slope, extending approximately 50 feet wide and 100 feet down the slope, to within approximately 50 feet of the stream. This area was vegetated with alder trees of relatively uniform diameter and height and appeared to be from 10 to 15 years old. The ground surface was otherwise somewhat barren. A pile of soil approximately 6 feet high and 20 to 30 cubic yards in volume was present against a bridge column on the northwest margin of the slide area (see Figure 2). Handhole HH-4 was advanced in this slide area and encountered soft to medium stiff silt and clay to the full depth explored of 7 feet, with ground water at approximately 4 feet deep. The southeast margin of the slide area had a side scarp up to 5 feet high. Southward from this side scarp, large, uniform-sized alder trees were present; this is evidence of previous slide activity.

3.6.3 **Eastern Slope**

The eastern slope was observed to have similar characteristics as the western slope. Although it does not have a wing wall and obvious recent slide areas adjacent to it as does the west end of the bridge, there is evidence of retreat of the top of the slope, south of the bridge, to approximately 25 feet east of the abutment. There was no sign of a previous bridge abutment beneath the east end of the bridge.

The top of the northeast slope has a down-dropped slide block, starting approximately 120 feet north of the bridge. It extends approximately 60 feet along a benched former access road. Adjacent to the bridge, a City sanitary sewer pump station is present at the top. Below the pump station, the slope is inclined generally at approximately 1.5H:1V and is vegetated with large alder trees (18 to 24 inches diameter), ivy, and sword ferns. The ground surface is hummocky, with local relief of 3 to 8 feet. Steep portions of the hummocky ground are inclined at up to 0.5H:1V. Handholes HH-1 and HH-2 were excavated on this slope. The lower 15 feet (vertical) of the slope has a shallow incline, ending at the incised stream channel. This portion is vegetated with underbrush and scattered alder.

The southeast slope has a broad, somewhat gently-sloped blackberry patch in the upper portion, evidence of ground disturbance (likely a slide). The yard edge of the house above has a steeply leaning, large-trunked deciduous tree with mature vertical regrowth, evidence of past slope failure up to the present edge of yard. Below the blackberry area, starting approximately 35 feet lower than top of slope, the slope increases in inclination to approximately 1.5H:1V to 1H:1V with local relief of 5 or so feet. Hard clay was exposed in an eroded rill approximately 10 feet deep, beneath a failed half-pipe stormwater drainage (with intact cast iron pipe above it). The upper portion of the rill had several feet of loose colluvial soil above the hard clay.

A shallow "apron" forming the lower 15 feet or so (vertically) of the slope is vegetated with underbrush and sparse alders. Many ground water seeps were evident, with overland flow to the stream.

4. CONCLUSIONS AND RECOMMENDATIONS

4.1 GENERAL

The soils underlying the project site are generally glacially consolidated in nature. However, significant evidence of past instability is present along most of the ravine side slopes. This past instability has resulted in the presence of colluvial soils and unfavorable, over steepened slope geometry. We expect that these conditions will result in continued slope instability and could affect the proposed bridge, if not mitigated. Due to the presence of observed evidence of past instability, we recommend that the proposed bridge be founded on drilled shaft foundations and not spread footings. The drilled shaft foundations should bear within the soils deep within the slope.

During demolition of the existing bridge structure, we recommend that the existing bridge foundations be cut off at 2 feet below ground surface and left in place. We do not recommend removal of the existing bridge pile foundations. Removal of these foundations could result in additional slope instability.

Construction of the bridge foundations may require construction of a temporary work trestle to access the interior pier locations. Due to the sensitivity of the existing slope, we recommend that driven pile or other vibration inducting foundations not be allowed for support of the work trestle. Therefore, drilled or screw-in foundations will be required to support any temporary work trestle. If a temporary work trestle is required, all foundations should remain in place after construction and be cut off 2 feet below grade. Removal of temporary foundations could result in future slope instability.

The near surface soils have the potential to experience landsliding during construction and lateral spreading at the base of the slope. The City and the design team will need to determine if they

want to implement physical mitigation measures or factor this potential instability into the design of the structure.

The current bridge abutments will include cast-in-place concrete wing walls. Additionally, instability predicted along Slope Stability Profile C-C' will require a soldier pile retaining structure to extend from the northwestern bridge wing wall. We expect that this wall will require tiebacks at portions of the wall and the soldier pile elements will need to extend to a sufficient depth to prevent landsliding.

Due to the presence of artesian groundwater conditions, and sloping ground surface at the interior pier locations, we recommend that the interior pier location drilled shafts be designed with permanent casing for the upper potions of the drilled shafts.

Given the unstable soils along the slope, it is possible that construction activities could further destabilize some areas, resulting is additional slope movements. We recommend that the contract documents include provisions for addressing slope movements that occur during and as a result of construction.

4.2 SEISMIC CONSIDERATIONS

4.2.1 Design Parameters

Earthquake loading for the proposed structure was developed in accordance with the General Procedure provided in Section 3.4 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*, 2nd Edition, 2011, and the Washington State Department of Transportation (WSDOT) amendments to the AASHTO *Guide Specifications* provided in the *Bridge Design Manual (LRFD)* (WSDOT, 2019). For seismic analysis, the Site Class is required to be established and is determined based on the average soil properties in the upper 100 feet below the ground surface. For this project, SPT blow counts obtained from the borings were utilized to classify the subject site as Seismic Site Class C. Therefore, Site Class C should be used with AASHTO seismic evaluations for this project.

The mapped seismic design coefficients for the design level event, which has a probability of exceedance of 7 percent in 75 years (equal to a return period of 1,033 years), were obtained using BridgeLink, a program developed by WSDOT to incorporate the probabilistic seismic hazard parameters from the *2014 Updates to the National Hazard Maps* (Peterson, et al., 2014) as well as adopt the site coefficients provided in ASCE 7-16. The recommended seismic coefficients for the design event are provided in Table 2. The spectral acceleration coefficient at 1-second period (S_{D1}) is between 0.3 and 0.5; therefore, Seismic Design Category C, as given by AASHTO Table 3.5-1 (AASHTO, 2011), should be used.

Table 2.
Seismic Coefficients Using AASHTO Guide Specifications
calculated by USGS Seismic Hazard Map

Site Class	Peak Horizontal Bedrock Acceleration PBA, (g)	Spectral Bedrock Acceleration at 0.2 sec S_s, (g)	Spectral Bedrock Acceleration at 1.0 sec S_1, (g)	Site Coefficients			Peak Horizontal Acceleration PGA (A_s), (g)
				F_{pga}	F_a	F_v	
C	0.403	0.912	0.269	1.200	1.200	1.500	0.484

4.2.2 Near Fault Ground Motion Considerations

As required by the AASHTO *Guide Specifications* near fault effects should be considered for bridges that are within 6 miles of a known fault. The Edgewater Creek Bridge is located about 2.4 miles northeast of the Southern Whidbey Island Fault Zone (SWIFZ). Given the proximity of the bridge site to the SWIFZ, near-fault effects should be considered for design analyses of the proposed bridge. The effects considered for this bridge include: (1) the large amplitude of the ground motions given the proximity to the fault, (2) potential for ground rupture, and (3) forward directivity. The large amplitude ground motions that could occur due to rupture of the Whidbey Island Fault are accounted for in the seismic design coefficients provided, since the *2014 National Hazard Maps* include the influence of the Whidbey Island Fault. Therefore, no additional consideration for large amplitude ground motions needs to be considered. The considerations for potential ground rupture and forward directivity are discussed in the following sections.

4.2.3 Ground Rupture

The site is located adjacent to the Whidbey Island Fault Zone, but there is no evidence from Lidar and gravity anomaly mapping that inferred fault traces may intersect the project site. Based on this information, we anticipate the likelihood of surface rupture at the project site to be low.

4.2.4 Forward Directivity

Structures located near faults experience the effect of forward directivity in which a short duration, high magnitude pulse-like motion is produced normal to the fault surface. Guidance from Chapter 6 of the WSDOT *Geotechnical Design Manual* (*GDM*) (WSDOT, 2019) indicates that directivity should be considered when the site is within 6 miles of a fault that is capable of producing a magnitude 5 earthquake or greater and directivity has not been incorporated into the probabilistic hazard maps that have been used. As the 2014 National Seismic Hazard Maps

(Peterson, et al., 2014) do not include directivity effects, the WSDOT *GDM* would recommend incorporating forward directivity into the design response spectrum. However, the AASHTO *LRFD Bridge Design Specifications* Section C3.10.2.2 (AASHTO, 2017) and the WSDOT *AASHTO Guide Specifications for LRFD Seismic Bridge Design Amendments* (WSDOT,2017) indicates the effects of directivity are normally only considered if the structure has a period greater than 0.5 seconds. AASHTO also indicates that forward directivity is normally only evaluated for essential or critical structures. Therefore, the need to incorporate forward directivity into the design response spectrum is dependent on the period of the bridge and critical nature of the structure.

If the fundamental period of the structure is greater than 0.5 seconds and/or the structure is considered an essential structure, then forward directivity should be incorporated into the response spectrum. In our experience, while the project site does not fall within the City of Seattle, the seismic retrofit philosophy presented within SDOT's publication of *Bridge Seismic Retrofit Philosophy, Policies, and Criteria* (BSRPPC), Revision 1, 2015 with amendments through 2018 is applicable to this project site. The SDOT BSRPPC recommends that forward directivity be accounted for by a 20 percent increase to the spectra obtained by the General Procedure for all periods greater than 1 second and tapers to 0 percent increase at 0.5 second for the design seismic events.

4.2.5 **Basin Effects**

Sedimentary basins are topographically low regions of underlying strong bedrock infilled with sediments that then became weak sedimentary rock. The geometry of these basins is often complex, and the formation of these structures can often be traced to a variety of sources. These basins have been shown to have varying effects on seismic waves and are known to significantly amplify ground motions during earthquakes, referred to as the Basin Effect. The amplification of seismic waves occurs as ground motions from a source project into a basin and reflect within the topographic bowl producing regions of constructive and destructive interference. These waves will often produce amplified surface ground shaking, generally increasing long-period motions above about 2 seconds.

At this time, consensus has not yet been reached on this topic to date. Due to this fact, it is our understanding that the City does not have a policy with respect to accounting for basin effects for bridges. Therefore, basin effects will not be considered for the Edgewater Creek Bridge project at this time.

4.2.6 **Liquefaction**

Liquefaction is a temporary loss of soil shear strength due to earthquake shaking. Loose, saturated cohesionless soils are the most susceptible to earthquake-induced liquefaction; however, recent experience and research has shown that certain silts and low-plasticity clays are

also susceptible. Primary factors controlling the development of liquefaction include the intensity and duration of strong ground motions, the characteristics of subsurface soils, in-situ stress conditions and the depth to ground water. Based on the WSDOT *GDM*, the liquefaction susceptibility of the soils along the project alignment was determined utilizing the simplified procedure originally developed by Seed and Idriss (1971) and updated by Youd et al (2001) and Idriss and Boulanger (2004, 2006).

The simplified procedure is a semi-empirical approach which compares the cyclic resistance ratio (CRR) required to initiate liquefaction of the material to the cyclic shear stress ratio (CSR) induced by the design earthquake. The factor of safety relative to liquefaction is the ratio of the CRR to the CSR; where this ratio is computed to be less than one, the analysis would indicate that liquefaction is likely to occur during the design earthquake. The CRR is primarily dependent on soil density, with the current practice being to base it on the Standard Penetration Test (SPT) N-value, corrected for energy consideration, fines content and earthquake magnitude. CSR is generally determined by the formulation developed by Seed and Idriss (1971) and relates equivalent shear stress caused in the soil at any depth to the effective stress at that depth and the peak ground acceleration at the surface.

The loose colluvial soils at the base of the ravine are susceptible to liquefaction, when saturated. Our explorations suggest that the liquefiable soil is likely constrained to the loose soil deposits found near the contact of the surficial colluvial deposits and the underlying, native transitional bed soils. Given the unpredictable nature of the previous failure scarps, it is difficult to constrain the depth and location that these liquefiable deposits may be encountered. Based on this understanding, our analysis indicated that a 2 to 4 foot layer of loose colluvial soils encountered below the ground water table and above the native transitional bed soils are likely to liquefy during a large earthquake, as shown on Figure 3.

4.2.7 **Liquefaction Settlement Analysis**

For liquefaction susceptible soil deposits, excess pore water pressure builds up during the earthquake excitation, leading to the loss of strength that occurs as a result of liquefaction. After the shaking stops, excess pore water pressures dissipate toward a zone where water pressure is relatively lower, usually the ground surface. The dissipation is accompanied by a reconsolidation of the soils (Ishihara and Yoshimine, 1992 & Tokimatsu and Seed, 1987). The reconsolidation is manifested at the ground surface as vertical settlement, usually termed as liquefaction-induced settlement or seismic settlement.

The magnitudes of potential liquefaction-induced settlement were evaluated using the methodologies developed by Idriss and Boulanger (2008), which are based on the relationship between cyclic stress ratio, corrected SPT blow counts, and volumetric strain. Using these methods, liquefaction-induced settlement is estimated to vary from 1 to 2 inches at the ground surface within the ravine and is likely to be differential in nature. While these deposits were only

encountered within the ravine during the final phase of our explorations, these deposits may be encountered as far up slope as the proposed western interior bridge pier.

We do not anticipate significant damage to occur to the proposed bridge structures due to vertical settlement as the drilled shafts will provide adequate bearing capacity extending to depths below this liquefiable layer. Potential down drag loads associated with this liquefaction are discussed in Section 4.7.3.

4.2.8   **Post Liquefaction**

Upon initiation of liquefaction, the shear strength of the liquefiable soils will be reduced to a residual shear strength while the excess pore pressure within the soil dissipates. For this project, residual shear strengths were estimated using a weighted average of the results of the Tokimatsu and Seed (1987), Seed and Harder (1990), Olson and Stark (2002), Idriss and Boulanger (2007) and Kramer (2008) relationships. The residual shear strengths were assigned as reduced friction angle materials and are estimated as a function of the equivalent clean sand SPT value, $(N_1)_{60cs}$, the potential for void redistribution, and the initial effective overburden stress. The residual shear strengths were then used to evaluate the potential for liquefaction induced slope failures beneath the bridge alignment and for the slopes perpendicular to the wing walls.

4.2.9   **Liquefaction Induced Slope Failures**

Liquefaction induced slope failures can either occur as lateral spreading or as a flow failure. Liquefaction induced lateral spreading occurs as the shear strength of liquefiable soils decrease during seismic shaking but does not decrease to the point that a complete flow failure would occur. Lateral spreading occurs cyclically when the horizontal ground accelerations combine with gravity to create driving forces which temporarily exceed the available strength of the soil mass. This is a type of failure known as cyclic mobility. The result of a lateral spreading failure is horizontal movement of the liquefied soils and any overlying crust of non-liquefied soils. Displacements associated with lateral spreading are generally quantifiable and on the order of a few to several feet. Lateral spreading is considered likely if the factor of safety of the slope under static loading using the post-liquefaction residual strengths is greater than 1.0.

In contrast, liquefaction induced flow failures result when the residual strength of the liquefied mass is not sufficient to withstand the static stresses that existed before the earthquake. Upon initiation of liquefaction induced flow failure, the liquefied soil behaves like a debris flow, characterized by very large displacements. Flow failures involve horizontal and vertical movements of the liquefied soils and any overlying crust of non-liquefied soils. The chaotic nature of flow failures is such that estimation of the magnitude of displacement is not reasonable. Flow sliding is likely if the stability (factor of safety) of the slope under static loading, using the post-liquefaction residual strengths, is less than 1.0. Our slope stability analyses performed for

the proposed improvements include consideration for liquefaction induced slope stability, which are presented in the Section 4.3.4.

4.3   GLOBAL SLOPE STABILITY EVALUATIONS

The stability of slopes in the vicinity of the bridge alignment were evaluated using limit equilibrium methods utilizing the computer program SLIDE 8.029 (Rocscience, 2019). Limit equilibrium methods consider force (or moment) equilibrium along potential failure surfaces. Results are provided in terms of a factor of safety, which is computed as the ratio of the summation of the resisting forces to the summation of the driving forces. Where the factor of safety is less than 1.0, instability is predicted. With limit equilibrium, the shear strength available is assumed to mobilize at the same rate at all points along the failure surface. As a result, the factor of safety is constant over the entire failure surface.

The model used in the slope stability analyses reflects three cross sections:

- Geologic Profile A-A' – This profile was generated along the centerline of the proposed bridge. The orientation of Geologic Profile A-A' is shown in Figure 2 and geometry is shown in Figures 3 and Appendix C – Figures C-1 through C-21.

- Slope Stability Profile B-B' – This profile was generated north of the wing walls outside of the influence of the bridge structure, as shown in Figure 2. The geometry of the profile is shown in Appendix D – Figures D-1 through D-4.

- Slope Stability Profile C-C' – This profile was generated along alignment shown in Figure 2 to evaluate the steepest portion of the western slope. The geometry of this profile is shown in Appendix E – Figures E-1 through E-11.

Each of these profiles were drawn using survey data of the existing ground surface and the locations of all the geotechnical explorations. Where appropriate, drilled shafts were added to the model at the boring locations to model the reinforcing added to the slopes from the drilled shafts. The shear strength of the drilled shaft elements was modeled based on the composite strength of the shafts and associated soil, in the vicinity of the shaft locations. The slope stability of each profile was evaluated using the material properties provided in Section 3.5. Results of our slope stability analysis are presented in Appendix C through E of this report. A detailed discussion of the slope stability modeling is provided below.

4.3.1 Static Slope Stability Analyses

The stability of each profile, under static loading conditions, was evaluated with Spencer's method and GME/Morgenstern-Price method using circular failure planes. Slope stability evaluations were completed under both the existing and proposed conditions, along each profile. For slopes supporting structures or roadways, factors of safety of 1.5, or greater, are desirable.

Static Global Stability – Existing Conditions

The factor of safety resulting from global static slope stability analysis, for each of the above existing condition profiles, are shown below in Table 3.

Table 3.
Global Slope Stability Analysis – Existing Conditions
Factor of Safety Under Static Loading Conditions

Slide Profile	Analysis Type	Structure	Slide Location	Loading Conditions	Figure Number	Factor of Safety
A-A'	Global Stability	Existing	Western Slope	Static	C-1	1.32
A-A'	Global Stability	Existing	Eastern Slope	Static	C-2	1.33
B-B'	Global Stability	Existing	Western Slope	Static	D-1	1.95
C-C'	Global Stability	Existing	Western Slope	Static	E-1	1.11

Our analyses indicated that, under existing conditions, all slope profiles evaluated possess factors of safety less than the required 1.5. Additionally, profile C-C' possesses a factor of safety of 1.11. This suggests that under the existing conditions, all evaluated slopes are deficient with respect to global slope stability, with profile C-C' being the most deficient slope.

Static Near Surface Stability– Existing Conditions

The factor of safety for a near surface slope stability analysis of each of the slope profiles, under static loading conditions, are shown below in Table 4.

Table 4.
Near Surface Slope Stability Analysis – Existing Conditions
Factor of Safety Under Static Loading Conditions

Slide Profile	Analysis Type	Structure	Slide Location	Loading Conditions	Figure Number	Factor of Safety
A-A'	Near Surface	Existing	Western Slope	Static	C-3	1.03
A-A'	Near Surface	Existing	Eastern Slope	Static	C-4	1.58
B-B'	Near Surface	Existing	Western Slope	Static	D-2	2.15
C-C'	Near Surface	Existing	Western Slope	Static	E-2	1.94

Near surface slope stability analyses indicate that the ravine side slopes currently possess varying levels of stability, under their current configuration. The colluvial soils under the bridge, along the western slope, represented by profile A-A', are shown to be barely stabile. This matches HWA's observations of past slope movement under the western portions of the bridge structure. As the near surface soils continue to undergo weathering and loosening due to freeze thaw cycles, we expect the stability of the near surface soils to decrease and near surface slope failures to occur across the ravine slide slopes, as has been seen in the past. Future slope failures will likely be triggered by large rain or snow events.

Static Global Stability – Proposed Conditions

For slope profiles where drilled shaft foundations are proposed, composite stiffness and cohesion parameters were computed, for use in the stability model, to account for the presence of the drilled shafts. Composite strength parameters were calculated using the method of weighted averages based on the proportional area of the drilled shaft with respect to the anticipated width of the bridge. Composite parameters were determined based on a 2.5-meter (8.2 foot) drilled shaft diameter, a single row shaft group configuration, a center-to-center spacing of 3 shaft diameters, and a shaft cap parameter extending the width of the existing bridge structure.

The factor of safety for a global slope stability analysis of each of the profiles, affected by the proposed bridge foundations, under static loading conditions, are shown below in Table 5.

Table 5.
Global Slope Stability Analysis – Proposed Conditions
Factor of Safety Under Static Loading Conditions

Slide Profile	Analysis Type	Structure	Slide Location	Loading Conditions	Figure Number	Factor of Safety
A-A'	Global Stability	Proposed Shafts	Western Slope	Static	C-5	1.81
A-A'	Global Stability	Proposed Shafts	Eastern Slope	Static	C-6	2.45
C-C'	Global Stability	Proposed Shafts	Western Slope	Static	E-3	1.29

These analyses indicate that the stability of the existing slopes, under static loading conditions, is increased due to the addition of the drilled shaft elements. Along profile A-A', along the centerline of the bridge, the addition of the drilled shafts constrains potential failure surfaces and increased the stability of the slope to above the required factor of safety of 1.5. Along profile C-C' the addition of the proposed drilled shafts increases the static factor of safety to 1.3 but does not increase it to the required factor of

safety of 1.5. Additional mitigation measures, to bring the static factor of safety along C-C' up to 1.5, will be required. These mitigation measures are discussed in Section 4.4.

Static Near Surface Stability– Proposed Conditions

The results of our near surface slope stability analysis, for the proposed condition, under static loading conditions, are shown below in Table 6.

Table 6.
Near Surface Slope Stability Analysis – Proposed Condition
Factor of Safety Under Static Loading Conditions

Slide Profile	Analysis Type	Structure	Slide Location	Loading Conditions	Figure Number	Factor of Safety
A-A'	Near Surface	Proposed Shafts	Western Slope	Static	C-7	1.68
A-A'	Near Surface	Proposed Shafts	Eastern Slope	Static	C-8	1.66
C-C'	Near Surface	Proposed Shafts	Western Slope	Static	E-4	1.94

With the introduction of drilled shaft elements, the near surface soils are constrained, improving the overall near surface stability of the slope beneath the bridge alignment. However, weathering of the near surface soils is expected to continue over the design life of the bridge structure. Therefore, near surface slope failures may occur in the vicinity of the bridge structure, over the design life of the structure. As a result, additional near surface slope stability mitigation measures may be necessary to stabilize the slope beneath the bridge for the design life of the structure.

4.3.2 Pseudo-Static Slope Stability Analyses

Each of the slope profiles were also evaluated using pseudo-static methods to evaluate the response of the slope under earthquake loading. Both Spencer's method and GMW/Morgenstern-Price's method were again used in this evaluation and both circular and non-linear failure planes were evaluated. Pseudo-static slope stability analyses model the anticipated earthquake loading as a constant horizontal force applied to the soil mass. For our analyses, we used a horizontal seismic coefficient of 0.242 g, which is one-half of the peak ground acceleration (PGA).

Pseudo-Static Global Stability – Existing Conditions

The factor of safety for a global slope stability analysis, of each of the existing slope profiles, under pseudo-static loading conditions, are shown below in Table 7.

Table 7.
Global Slope Stability Analysis – Existing Conditions
Factor of Safety Under Pseudo-Static Loading Conditions

Slide Profile	Analysis Type	Structure	Slide Location	Loading Conditions	Figure Number	Factor of Safety
A-A'	Global Stability	Existing	Western Slope	Pseudo-static	C-9	0.85
A-A'	Global Stability	Existing	Eastern Slope	Pseudo-static	C-10	0.91
B-B'	Global Stability	Existing	Western Slope	Pseudo-static	D-3	1.31
C-C'	Global Stability	Existing	Western Slope	Pseudo-static	E-5	0.87

These pseudo-static factors of safety are less than the minimum required pseudo-static factor of safety of 1.1 for all profiles except Profile B-B'. This analysis indicates that global slope instability near the existing bridge piers is likely to occur, under current conditions, during the design earthquake.

Pseudo-Static Near Surface Stability – Existing Conditions

The factor of safety for a near surface slope stability analysis of each of the slope profiles, under pseudo-static loading conditions, are shown below in Table 8.

Table 8.
Near Surface Slope Stability Analysis – Existing Conditions
Factor of Safety Under Pseudo-Static Loading Conditions

Slide Profile	Analysis Type	Structure	Slide Location	Loading Conditions	Figure Number	Factor of Safety
A-A'	Near Surface	Existing	Western Slope	Pseudo-static	C-11	0.70
A-A'	Near Surface	Existing	Eastern Slope	Pseudo-static	C-12	1.09
C-C'	Near Surface	Existing	Western Slope	Pseudo-static	E-6	1.39

Near surface slope stability analyses indicate that both side slopes of the ravine for Profile A-A' are less than the minimum required pseudo-static factor of safety of 1.1, under the current configuration. The colluvial soils under the bridge, along the western slope, represented by profile A-A', are shown to be barely stable. This analysis indicates that global slope instability near the existing bridge piers is likely to occur, under current conditions, during the design earthquake.

Pseudo-Static Global Stability – Proposed Conditions

Proposed slope conditions were analyzed utilizing the composite stiffness and cohesion parameters to account for the presence of the drilled shafts. The factor of safety for a global slope stability analysis of each of the proposed slope profile, under pseudo-static loading conditions, are shown below in Table 9.

Table 9.
Global Slope Stability Analysis – Proposed Condition
Factor of Safety Under Pseudostatic Loading Conditions

Slide Profile	Analysis Type	Structure	Slide Location	Loading Conditions	Figure Number	Factor of Safety
A-A'	Global Stability	Proposed Shafts	Western Slope	Pseudo-static	C-13	1.30
A-A'	Global Stability	Proposed Shafts	Eastern Slope	Pseudo-static	C-14	1.64
C-C'	Global Stability	Proposed Shafts	Western Slope	Pseudo-static	E-7	1.02

The pseudo-static factors of safety beneath the bridge alignment are greater than the minimum required pseudo-static factor of safety of 1.1. This analysis indicates that with the introduction of the drilled shafts, global slope instability parallel to the bridge alignment is not expected to occur. However, our analysis indicates that the pseudo-static factor of safety along Profile C-C', the steepest profile, is less than the required factor of safety of 1.1. This suggest that landsliding along Profile C-C' could occur as a result of the design earthquake. Instability along this profile could reduce the capacity of the bridge foundations and would result in collapse of the associated northwestern wing wall. Therefore, additional slope stability mitigation measures will be required to address the deficient stability along Profile C-C'.

Pseudo-Static Near Surface Stability – Proposed Conditions

The results of our near surface slope stability analysis, for the proposed condition, under pseudo-static loading conditions, are shown below in Table 10.

Table 10.
Near Surface Slope Stability Analysis – Proposed Condition
Factor of Safety Under Static Loading Conditions

Slide Profile	Analysis Type	Structure	Slide Location	Loading Conditions	Figure Number	Factor of Safety
A-A'	Near Surface	Proposed Shafts	Western Slope	Pseudo-static	C-15	1.12
A-A'	Near Surface	Proposed Shafts	Eastern Slope	Pseudo-static	C-16	1.12
C-C'	Near Surface	Proposed Shafts	Western Slope	Pseudo-static	E-8	1.41

With the introduction of drilled shaft elements, the near surface soils are constrained, improving the overall near surface stability of the slope beneath the bridge alignment. However, weathering of the near surface soils is expected to continue over the design life of the bridge structure. Therefore, near surface slope failures may occur in the vicinity of the bridge structure, over the design life of the structure. As a result, additional near surface slope stability may occur over the design life of the structure as a result of design level earthquakes.

4.3.3 Profile C-C' Slope Stability Mitigation Measures

Our static and pseudo-static stability analysis indicate that profile C-C' will not possess the required minimum factors of safety for slope stability under the proposed condition. These deficient factors of safety could result in the occurrence of slope instability that could negatively affect the bridge structure and associated northwestern wing wall. To mitigate the potential for slope instability in this area, we recommend that the northwestern wing wall be constructed as a soldier pile and lagging wall, rather than a conventional Structural earth wall (SEW).

Due to the potential for near surface instability, this soldier pile wall will need to account for the potential loss of passive pressures at the toe of the wall due to near surface slope failures down slope of the wall. Recommendations associated with this proposed structure are provided in Section 4.4.

For slope stability analysis completed along the proposed wall alignment, composite stiffness and cohesion parameters were computed for the wall elements. Composite strength parameters were calculated using the method of weighted averages based on the anticipated spacing of the vertical soldier pile elements. Composite parameters were determined based on a 3-foot pile diameter and a center-to-center spacing of 8-feet.

The factors of safety, for our global slope stability analysis, with the inclusion of this retaining structure along the critical alignment, under pseudo-static loading conditions, are shown below in Table 11.

Table 11.
Global Slope Stability Analysis – Proposed Stability Mitigation Measures
Factor of Safety Under Pseudostatic Loading Conditions

Slide Profile	Analysis Type	Structure	Slide Location	Loading Conditions	Figure Number	Factor of Safety
B-B'	Global Stability	Proposed Retaining Structure	Western Slope	Pseudo-static	D-4	1.59
C-C'	Global Stability	Proposed Retaining Structure	Western Slope	Pseudo-static	E-9	1.47

With the recommended mitigation measures, the pseudo-static factors of safety of profile C-C' and B-B' are greater than the minimum required pseudo-static factor of safety of 1.1. This analysis indicates that with the introduction of the bridge drilled shaft foundations and soldier pile wing wall, global slope instability along profile C-C' is mitigated.

4.3.4 Post Liquefaction Slope Stability Analysis

Each of the slope profiles were also evaluated under post liquefaction conditions. Post liquefaction conditions were modeled by assuming static loading and reduced residual strength parameters for the liquefiable soil layers. Liquefiable soils were only encountered near the base of the western slope; therefore, post liquefaction slope stability analysis was only conducted in this area. Both Spencer's method and GMW/Morgenstern-Price's method were again used in this evaluation and a circular and non-linear failure plane passing through this weakened, liquefied soil was evaluated.

Post Liquefaction Stability Analysis – Existing Conditions

The factor of safety for slope stability analysis of each of the existing slope profiles, under post liquefaction loading conditions, are shown below in Table 12.

Table 12.
Global Slope Stability Analysis – Existing Conditions
Factor of Safety Under Post Liquefaction Loading Conditions

Slide Profile	Analysis Type	Structure	Slide Location	Loading Conditions	Figure Number	Factor of Safety
A-A'	Global Stability	Existing	Western Slope	Post LQ	C-17	1.53
A-A'	Near Surface	Existing	Western Slope	Post LQ	C-18	0.80
C-C'	Global Stability	Existing	Western Slope	Post LQ	E-10	1.46

These analyses indicate that, under existing conditions, we would expect the initiation of liquefaction to result in near surface slope instability near the base of the ravine. Given a factor of safety less than 1.0, we would expect this instability to manifest as flow sliding. Flow sliding would result in significant movement of the near surface soils, near the base of the ravine. These movement would reduce support for soils higher up the ravine and likely cause additional slope failures.

Post Liquefaction Stability Analysis – Proposed Conditions

Proposed slope conditions were analyzed, under post liquefaction conditions, utilizing the composite stiffness and cohesion parameters to account for the presence of the drilled shaft foundations. The factor of safety for a global slope stability analysis of each of the proposed slope profiles, under post liquefaction loading conditions, are shown below in Table 13.

Table 13.
Global Slope Stability Analysis – Proposed Conditions
Factor of Safety Under Pseudostatic Loading Conditions

Slide Profile	Analysis Type	Structure	Slide Location	Loading Conditions	Figure Number	Factor of Safety
A-A'	Global Stability	Quarry Spalls	Western Slope	Post LQ	C-19	1.51
A-A'	Global Stability	Proposed Shafts	Western Slope	Post LQ	C-20	3.42
A-A'	Near Surface	Proposed Shafts	Western Slope	Post LQ	C-21	1.16
C-C'	Global Stability	Proposed Retaining Structure	Western Slope	Post LQ	E-11	2.88

These analyses indicate that the introduction of the proposed drilled shaft foundations generally reduce the potential for post liquefaction instability. However, near surface instability is still expected to occur near the base of the ravine, as a result of liquefaction. With the introduction of the proposed bridge foundations, this instability is expected to manifest as lateral spreading of the soil in the vicinity of the western interior pier. As these soils and the overlying non-liquefiable soils spread laterally, they are expected to apply large lateral loads to the interior pier foundations. Recommendations associated with addressing potential lateral spreading, at the base of the ravine, are provided in Section 4.5.

4.3.5 **Slope Stability Summary**

Our stability analysis indicates the existing slope soils, under the current condition, are generally stable, with respect to deep seated global slope failures. However, the slopes are prone to near surface slope failures under static loading. Under seismic loading, portions of the existing slopes are expected to fail. Introduction of the drilled shaft foundation elements, at the abutments and within the ravine slope, will improve the static and pseudo-static stability of the slopes directly under the bridge. However, our analyses indicate that the critical western over-steepened slope, represented by stability profile C-C', is expected to be deficient under static loading conditions and unstable under pseudo-static loading conditions, even with construction of the bridge foundations. Instability along this cross section could result in damage to the bridge structure and the associated northwestern wing wall, resulting in a threat to life safety of the traveling public. Therefore, stability mitigation will be required to stabilize the soils along stability profile C-C'. Recommendations associated with stability mitigation along profile C-C' are provided in Section 4.4.

While construction of the bridge foundations and mitigation measures, near the northwestern corner of the bridge, will generally stabilize the slopes under static and psuedo-static loading, post liquefaction instability is expected at the base of the ravine. The onset of post liquefaction instability is expected to result in the application of large lateral spreading loads on the western interior pier foundations. The bridge design will need to account for the potential for post liquefaction instability at the base of the ravine. Details associated with addressing post liquefaction instability are provided in Section 4.5.

While our slope stability analyses suggests that the construction of the drilled shaft foundations and retaining structure will generally stabilize the slopes in the vicinity of the bridge, the potential for slope instability to occur during bridge construction is a possibility. If slope instability does occur during construction, slope repaires will be required. Recommendations associated with addressing potential construction related slope instability are provided in Section 4.6.

4.4   NORTHWESTERN WING WALL SLOPE MITIGATION

Our slope stability analyses indicate that slope stabilization measures, in addition to construction of the proposed bridge foundations, will be required to stabilize the critical slope below the northwestern corner of the proposed bridge structure. HWA has modeled slope stabilization methods ranging from slope reconstruction to installation of an anchored slope retention system. Our modeling suggested that the most cost effective and constructable slope stabilization measure would be to construct the proposed northwestern wing wall as a soldier pile and lagging wall. Construction of a soldier pile wall at this location would constrain any future slope failures to down slope of the wall alignment. Our preliminary modeling suggests that the soldier pile vertical element would require a minimum 50 foot of embedment to adequately constrain future landsliding and protect the bridge foundations. Installation of soldier pile vertical elements should be limited to drilling methods to avoid vibrations that could further destabilize the slope. We expect that the soldier pile wall will require tiebacks, as some soil at the toe of the wall is expected to slide away over the design life of the wall. Specific recommendations associated with the recommended soldier pile wall are provided in Section 4.8.1.

4.5   ADDRESSING LIQUEFACTION INDUCED SLOPE INSTABILITY

As indicted in Section 4.3.4, post liquefaction induced slope instability is expected to occur at the base of the ravine. This instability is expected to manifest in the form of lateral spreading that would result in large lateral loads applied to the western interior bridge pier foundations. Post liquefaction instability in the vicinity of the western interior pier can be addressed through either mitigation to stop the instability or through designing the bridge foundations to resist the predicted loading associated with the instability. Each of these options is discussed below.

4.5.1 Mitigation to Stop Liquefaction Included Instability

Many options are available to mitigate the onset of liquefaction and liquefaction induced instability. These generally consist of some form of ground improvement to stop liquefaction from occurring. However, the location of the potentially liquefiable soils is such that mobilizing ground improvement equipment would likely be cost prohibitive. Therefore, if mitigation of the liquefaction induced instability is desired, we would recommend removal of the existing liquefiable soils and replacement with 4-8 inch quarry spalls. This over excavation of loose material should extend to the underlying, medium dense to dense, native Transitional Bed soils and extend at least 50 feet from the proposed ravine bridge foundation element towards to the stream. An approximation of this estimated excavation has been provided as a plan view and cross section in Figures 4 and 5, respectively. For construction estimation purposes, we anticipate approximately 1,700 yards of material will need to be removed and replaced. This estimation is a rough approximation and has been estimated based on our anticipated depth of soft, liquefiable soils. Additional material may be removed and replaced if the extent of liquefiable soils encountered is greater than anticipated.

We expect that the excavation could be accomplished with conventional excavating equipment such as backhoes and excavators. To limit potential groundwater seepage into the excavation, we would recommend that the excavation take place during the dry summer months and not extend into the wet season. The base of the excavation is expected to extend below the ground water table. To limit caving an initiation of slope movements, we recommend that the over excavation be completed in small section that are backfilled prior to excavating the next section. The depth of the excavation may extend to a depth of approximately 15 feet bgs. Horizontal benches should be at least 5 feet wide or the width of the compaction equipment, whichever is wider.

We expect that over excavation and replacement of the potentially liquefiable soils will require construction of temporary access to the base of the ravine. We would recommend that the design team develop a temporary construction access concept for getting tracked equipment to the base of the ravine. Although the contractor may not choose to utilize the access concept developed by the design team, providing a valid concept for bidding is recommended. We expect providing temporary access across the steep slopes will require the construction of temporary walls and removal and replacement of loose, near surface material. Due to the loose nature of the near surface colluvium along likely access routes, we recommend that the design team consider the use of temporary soldier pile walls, along portions of the access path, to provide safe access for the contractor.

4.5.2 **Design Foundations to Resist Lateral Spread Loads**

If implementation of the above-described liquefaction induced instability mitigation measures is not desirable, the team could allow the expected liquefaction induced instability to occur and design the structure to resist the anticipated loading.

During the anticipated lateral spreading event, the liquefiable soils are expected to incrementally move down slope towards the creek, transporting any overlying crust of non-liquefiable soils. For design purposes, the pressure exerted on the shafts from the liquefiable soil flowing past the shaft is estimated as 30 percent of the confining pressure, as presented by the Japanese method referenced in the WSDOT *GDM*. The pressure exerted on the shafts from non-liquefiable soils that are transported during the lateral spread event was calculated using the full passive pressure of the non-liquefiable soil. Liquefiable soils were only encountered near the proposed foundation element for the western interior pier; therefore, lateral spread loading diagrams are only provided for this pier location. The anticipated lateral spread loading diagram, for this interior pier, is provided on Figure 10. We expect that the saturated colluvium will liquefy and begin to mobilize early in the design earthquake. Therefore, we recommend that the lateral spread loads be combined with 100 percent of the bridge inertial loads.

The lateral spread pressures are provided per one foot of width and must be multiplied by the appropriate width factor, as provided in the note on the figure. Note that no softening of the

earth pressures was applied at the contacts between liquefied and non-liquefied material. For transitioning, we followed the recommendation by Ashford et al. (2011) to not include the effect of softening for large diameter shafts.

4.6   SLOPE STABILITY DURING CONSTRUCTION

Our slope reconnaissance indicated the presence of very loose to loose colluvium is evidence of past slope failures. This colluvium likely developed because of mechanical weathering, gradual slope creep over time, and past slope failures. It is our experience that once the near surface colluvium on similar slopes reaches thicknesses more than about 3 feet, they are prone to sliding during periods of heavy rainfall or unusually wet weather. These failures generally consist of the loosened, near surface colluvium sliding down the slope and exposing the underlying dense glacial soils. Given the thickness of the colluvium observed during our slope reconnaissance, slope failures could occur during or as a result of construction activities.

In the event that a slope failure occurs during construction, they could impact the contractor's operations and/or adjacent properties. Repair of unanticipated slope failures, that occur during construction, can be costly, result in significant construction delays, and increased City liability. Addressing the potential for slope instability during construction can be accomplished by implementing mitigation measure to prevent slope failures during construction or by structuring the contract to have adequate funds to repair slope failures if they occur during construction.

4.6.1 Implement During Construction Slope Mitigation Measures

To reduce the risk of slope failures during construction, the design team could choose to implement mitigation measures that stabilize the near surface soils along the slopes in the vicinity of the bridge alignment. Given the thickness of the colluvium along the existing slopes, over excavation and replacement of the loose colluvial soils is expected to be the most cost-effective mitigation measure to reduce potential, during construction slope failures. We expect that excavation and replacement of the colluvial soils would occur at the same time as removal of the demolished bridge structure. Once the bridge structure debris is removed, the contractor would continue to excavate the poor-quality colluvium, down to the competent glacially consolidated soils. The excavation of the colluvium would be completed in small section and backfilled prior to moving to the next section. This would be done to avoid initiating larger landsliding through opening a large excavation prior to backfilling. Excavated colluvium would be replaced with angular, high shear strength, aggregate. The extent of the over excavation and replacement would extend approximately 10 feet past the outside edge of the proposed bridge structure and from the bottom of the bridge abutment to the base of the slope or wetland boundary, on both sides of the ravine.

Removal of bridge demolition debris and the associated over excavation and replacement will require the construction of access to the base of the ravine. We expect that this will require

removal of colluvial soils and installation of temporary walls along the alignment of the access path.

Over excavation and replacement of the pseudo-stable colluvial material would greatly reduce the potential for slope instability during or as a result of construction activities. However, it should be noted that even with implementation of over excavation and replacement of the colluvium under the bridge, instability of the colluvium outside the bridge right-of-way is possible during construction. Therefore, even with the implementation of slope mitigation measures, we would recommend that the contract documents include a force account bid item to address stability issues during construction.

4.6.2 **Address Slope Instability as it Occurs**

In the event that implementation of the above-described slope mitigation measures is considered too costly, the City could choose to not implement the excavation and replacement of the colluvium and just address slope instability as it occurs. Under this option, the contact documents would include a large force account bid item for repairing slope failures during construction. If no slope failures occur during construction, the bid item would not be used. However, if slope failures do occur during construction, they will likely be more costly to repair than if the mitigation measures were implemented at the start of construction. The increased cost would be due to the likelihood that slope failures during construction would impact the contractors progress and/or construction methods.

4.6.3 **Pre-Construction Survey**

Regardless of which option for addressing potential during construction slope instability the City chooses, HWA would recommend that the City implement a detailed preconstruction survey of the surrounding slopes and homes. This survey should include detailed documentation of all existing slope cracks, past landslides, and house foundations. We would recommend that this survey extend at least 300 feet up and down stream of the bridge project. Completing this preconstruction survey will establish a documented baseline of the slope and private property conditions prior to construction. In the event that private property owners claim that construction activities caused instability, this preconstruction survey would go a long way to protect the city from false or questionable claims.

4.7 **BRIDGE FOUNDATIONS**

We recommend that the bridge abutments be founded on drilled shaft foundations that bear in the very dense Whidbey Formation at both abutment locations and within the ravine. To limit potentially damaging vibrations, we recommend that the drilled shafts be installed using the oscillating casing method. We understand that the equipment to be used to install these foundations are based in metric and that these foundations will consist of 1.5-, 2-, 2.5-, or 3-

meter diameter drilled shafts. Additionally, the abutment foundations shaft configuration has not been determined at this time. For the purpose of our design, we have assumed that the proposed top of shaft elevation for the eastern bridge abutment will be 120 feet Above Mean Sea Level (AMSL) and the top of shaft elevation for the western abutment will be 125 feet AMSL. Additionally, we have assumed an approximate elevation of 62 feet and 64 feet AMSL for the western and eastern interior piers, respectively. All foundations recommendations provided below assume that the bridge foundations are designed to resist the anticipated lateral spread loads provide in Section 4.5.2 and no over excavation and replacement of liquefiable soils is completed. If the team decided to mitigate liquefaction induced instability, through over excavation and replacement of liquefied soils, HWA should be notified and allowed to update our foundation recommendations.

4.7.1 **Drilled Shaft Axial Capacity**

Axial shaft capacities for drilled shaft foundations were evaluated using LRFD methods in general conformance with the procedures referenced in the FHWA Drilled Shafts Manual, 8th Edition (Brown, et al., 2018). This method provides a revised method to the Reese and O'Neill method (1989). Axial shaft capacities will be derived from both shaft friction and end bearing. Nominal axial shaft capacities versus embedment depths for the eastern and western abutments as well as the two support structures within the ravine are presented in Figures 6 through 9, for each of the proposed diameter shafts. As indicated on these figures, resistance factors (ϕ) of 0.55 and 0.45 should be applied to the nominal side resistance, for the Strength I Limit State for cohesionless and cohesive soils, respectively. Resistance factors of 0.5 and 0.4 should be applied to the nominal base resistance for Strength I Limit State design for cohesionless and cohesive soils, respectively. For the Extreme I and the Service I Limit States, the resistance factor ϕ should be 1.0 for both side and base resistance.

It should be noted that the friction capacity at each foundation location was neglected from the existing ground surface to a depth of 5 feet to account of unraveling of near surface soils or loosening of material due to construction.

For the Service I Limit State, total shaft resistance (i.e., friction plus end bearing) is provided for an allowable settlement of 1 inch. If a Service I Limit State capacity for a different settlement value (e.g. 2 inches or ½ inch) is needed, we should be contacted to revise our calculations. Additionally, we recommend that the shafts be spaced no closer than 3 shaft diameters to avoid excessive reductions in vertical capacity due to group affects.

4.7.2 **Vertical Capacity Group Reduction Factors**

Placement of a drilled shaft less than three shaft diameters from an existing shaft reduces the effective stresses against both the side and base of the existing shaft. As a result, the capacities of individual drilled shafts within a group tend to be less than the corresponding capacities of

isolated shafts. For a single row configuration, the group reduction factor approached a value of 0.9 as the center-to-center spacing approaches two shaft diameters.

Table 14 illustrates the minimum center-to-center spacing that should be utilized for each drilled shaft diameter size in order to maintain a group reduction factor (η) of 1.0.

Table 14.
Estimated Group Reduction Factors for Various Drilled Shaft Diameters

Drilled Shaft Diameter (meter)	Center-to-Center Spacing (feet)	Shaft Diameter Spacing Ratio
1.5 (4.92 feet)	15	4.06
2 (6.56 feet)	20	3.05
2.5 (8.20 feet)	25	3.66
3 (9.84 feet)	30	3.05

If the center-to-center spacing for the proposed foundation is anticipated to be less than the values presented in Table 14, we should be notified, and the appropriate group reduction factor should be provided.

4.7.3 Down Drag Loading Parameters

Down drag loading on shafts occurs when the surrounding soil settles or otherwise moves downward relative to the shaft. Downward movements on the order of ¼ inch are sufficient to fully mobilize negative shaft resistance or down drag.

Downdrag loads will be imposed on the shafts near boring BH-3 during the post-liquefaction condition due to liquefaction-induced settlement. The anticipated downdrag loads for the loading condition at this pier is provided in Table 15 for each of the proposed diameter shafts. Downdrag loads due to liquefaction should be applied to the Extreme I Limit State with a load factor of 1.05 (Allen, 2005).

Table 15: Estimated Downdrag Loads due to Liquefaction
at Boring BH-3 for Various Diameter Shafts

Diameter of Shaft	Downdrag due to Liquefaction
1.5 meters	85 kips
2.0 meters	110 kips
2.5 meters	145 kips
3.0 meters	160 kips

4.7.4 **Drilled Shaft Lateral Design Parameters**

The proposed drilled shafts will extend into the very dense, glacially-consolidated soils. We understand that the design team desires to use conventional p-y method of lateral analysis (i.e., LPILE) to estimate shears, moments and deflections of the shafts. Soil parameters for use in LPILE analyses are provided in Appendix D. The soil parameters provided in Appendix D may be used with LPILE for lateral structural analysis and design of the abutments. Parameters are provided for static, non-liquefied analyses.

Research indicates that deep foundations constructed on nearly level ground and within 4 pile or shaft diameters of where a slope begins its downward accent will experience reductions in the lateral capacity of the foundation element (Barker, 2012). Based on this research, we recommend that zero lateral resistance be assumed for the abutments drilled shafts from the top of the proposed shafts to a depth at which the shaft face is 4 shaft diameters away from the surface exposure of the slope. This recommendation is reflected in the provided LPILE tables for each of the foundation locations.

The p-y curves generated by the lateral parameters provided in Appendix D must be modified by the applicable p multipliers to account for the group reduction effects. The p multipliers for shafts spacing of 3 shaft diameters (e.g. 2.5-meter diameter shafts on 25-foot spacing) are provided in Table 16. If center-to-center spacings for the proposed foundation are anticipated to differ from those presented in Table 16, we should be notified, and the design charts should be reviewed and modified as necessary.

Table 16.
P Multipliers for Center-to-Center Spacing of 3 Shaft Diameters

Row	P Multiplier
1	0.7
2	0.5
3 or more	0.35

The same p multiplier factor should be applied parallel and perpendicular to the group shaft alignment.   The following diagram shows how the p multipliers should be assigned with respect to the load direction and shaft orientation.

Parallel Direction

Perpendicular Direction

4.7.5 **Drilled Shaft Construction Considerations**

The drilled shafts will be drilled through loose to medium dense fill, loose/soft colluvium, hard fine-grained Transitional Bed deposits, and will likely terminate in very dense coarse-grained Whidbey Formation deposits. Given the proximity of the proposed drilled shafts to the existing steep slope, we recommend that the oscillating casing method of shaft construction be used to limit vibrations that could affect the steep slope and the adjacent residential neighborhood. It is likely that some temporary support elements will be required to provide the oscillator with adequate bearing capacity to extract the temporary casing after the shaft is complete. We recommend that these temporary supports be anticipated by the design team but designed by the contractor.

Although not encountered in our geotechnical borings, the subsurface soils may contain cobbles and boulders. Per the Unified Soil Classification System (USCS), cobbles are defined as a rock with a dimension between 3 and 12 inches; boulders are defined as rock with a dimension greater than 12 inches. The drilled shaft contractor should be prepared to encounter and handle cobbles and boulders.

The abutment shafts will extend below a perched groundwater table, and into an artesian aquifer which will be encountered between elevations 70 and 60 feet AMSL. The contractor should be prepared to mitigate the wet conditions and flowing artesian conditions. During construction the contractor should maintain 30 feet of head above the excavation depth to account for the artesian pressure. Once below the water table, the drilling spoils excavated from the shafts will be saturated. These soils will need to be transported to a nearby facility for decanting or be loaded into special sealed dump trucks for transport off site.

Given encountered artesian groundwater conditions, the upper portions of the drilled shaft foundations should be designed with permanent casing for each interior pier drilled shaft. This casing will need to be expected deep enough to prevent blow out of the shaft concrete during construction of the drilled shafts and allow for the maintenance of a water head to offset artesian pressures. Further details pertaining to casing depths will be provided once the design progresses.

Due to the observed artesian pressures, we recommend that the drilled shaft specifications require the contractor to implement, at no additional cost, all necessary measures to ensure proper shaft installation and concrete curing under the observed artesian pressures and soil conditions. This should include but not be limited to installation of dewatering to allow shaft concrete to cure, if required.

4.7.6 **Temporary Work Trestle**

It is our understanding that construction of the proposed bridge foundations may require construction of a temporary work trestle to access the interior pier locations. If this option is utilized, the design and construction of this temporary work trestle should be the responsibility of the prospective contractor.

Due to the sensitivity of the existing slope, we recommend that driven pile or other vibration inducting foundations not be allowed for support of the work trestle. Therefore, drilled or screw-in foundations will be required to support any temporary work trestle.

The proposed foundation for this temporary work trestle should take into consideration the placement of near surface slope mitigation solutions, such as those discussed in Section 4.5.1, as drilling through quarry spalls or other aggregate may prove to be difficult. Additionally, if a temporary work trestle is required, all foundations should remain in place after construction and be cut off 2 feet below grade. Removal of temporary foundations could result in future slope instability.

4.7.7 **Abutment Lateral Loading**

Design lateral earth pressures for abutment walls assume that the walls are backfilled with properly compacted Structural Fill, as described in Section 5.1. It should be assumed that the walls will be free to deflect by at least 0.001H, where H is the retained height of the wall, to allow active conditions to develop. Using these assumptions, an equivalent fluid pressure of 35 pounds per cubic foot can be assumed for static loading condition. Under earthquake loading conditions, the retaining wall is also anticipated to yield an adequate amount to allow development of active conditions. The evaluation of the active pressures experienced during a seismic event can be approximated using the Mononobe-Okabe method utilizing 0.5 times the PGA for the site that yields 0.242 g. For design purposes, a design active-plus-seismic equivalent fluid pressure of 56 pounds per cubic foot may be assumed. These earth pressures assume no accumulation of water behind the wall. Proper wall drainage should be constructed to ensure that hydrostatic pressures do not develop behind the wall structure.

We recommend that that passive pressure in front of the abutment wall be neglected, assuming soils move away from the wall over the design life of the structure, and resistance the above described loading be provided by the lateral capacity of the drilled shaft foundations.

4.7.8 **Bridge Wing Walls**

We understand that wing walls will be required at all four corners of the bridge structure to support the bridge approach slabs. Based on the provided plan sets, we understand that the walls will consist of cast-in-place cantilevered concrete system. The length of each wall is currently anticipated to be 15 feet. At the northern corner of the western bridge abutment, the wing wall

will tie into additional retaining wall structures that will be required to facilitate roadway widening. We anticipate this retaining wall structure will consist of a soldier pile wall with tiebacks.

For the northwestern wing wall, we recommend that the cast in place portion of the wing wall, that is structurally connected to the abutment, be reduced in length to no greater than 5 feet. This recommendation to shorten the CIP portion of the northwestern wing wall is to allow for the recommended soldier pile wall to extend closer to the abutment and increase the stability of the roadway. Further discussion of the northwestern soldier pile wall is provided in Section 4.8.1.

Wing Wall Lateral Earth Pressures

Design lateral earth pressures for the wing walls should be assumed to be the same as those provided for the bridge abutment walls, as described in Section 4.7.7. Due to near surface slope failures, observed across the ravine side slopes, further unravelling of near surface soils, in front of the proposed wing walls, may occur during the design life of the bridge. As a result, we recommend that that passive pressure in front of the proposed wing walls be neglected completely for wing wall design. Therefore, in order to mitigate overturning and sliding failures, we recommend that the proposed wing walls be structurally connected to the approach abutments and wing wall foundations designed to resist anticipated loading without passive pressure acting on the front of the wall. Unconventional foundation geometries such as oversizing the L-wall configuration may be utilized in order to resist the anticipated loading without passive pressure acting on the toe of the walls.

Wing Wall Bearing Capacity

Based on the results of boring BH-1 and a proposed footing foundation Elevation of 110 feet AMSL, we anticipate that the northeastern and southeastern walls will bear on medium stiff fill and colluvial soils and can be designed for an ultimate bearing capacity of 2,000 pounds per square foot (psf) to resist overturning moments. Based the results of boring BH-4 and a proposed footing foundation Elevation of 115 feet AMSL, we anticipate that the northwestern and southwestern wing walls will bear on medium dense fill soils and can be designed for an ultimate bearing capacity of 3,000 pounds per square foot (psf) to resist overturning moments.

If greater bearing capacities are required, footing can be supported on small diameter drilled shafts. Further recommendations associated with shaft supported wing walls can be provided if required.

4.8   RETAINING WALLS

We understand that additional retaining walls will be extended beyond the extents of the bridge wing walls at the northwestern and southwestern walls to facilitate grade changes associated with the bridge structure. Based on our global slope stability analysis, the northwestern retaining wall

(running parallel with Mukilteo Lane) will be designed to provide additional slope stabilization, as discussed in Section 4.4. We anticipate that this wall will consist of a soldier pile wall.

The southwestern retaining wall (the side access road leading south to LaMar Drive) will be designed as a cut wall. A soldier pile wall configuration is expected to be the best wall solution at this location due to right-of-way restrictions.

4.8.1 **Northwestern (Mukilteo Lane) Abutment Wall**

Per the discussion provided in Section 4.4, it is our recommendation that the slope at the northwestern corner of the bridge be stabilized with a soldier pile and lagging wall. We recommend that the CIP wing wall at this location be shortened to a maximum length of 5 feet and the soldier pile wall start immediately at the end of the shortened CIP wing wall.

Soldier pile wall and lagging systems rely on embedment below the retained portion of the wall and tieback anchors to support the lateral earth pressures exerted by the retained soil. For soldier pile and lagging wall systems, steel H-piles are generally placed in drilled shafts, spaced at approximately 6- to 8-foot centers. The diameter of typical soldier pile shaft excavations is on the order of 2 to 3 feet and the H-piles are imbedded below the bottom of the excavations. Once the H-piles are installed, the drilled shafts are filled with concrete or controlled density flowable fill (CDF). If tiebacks are required, they are drilled into place and stressed after the soldier piles are installed. Typically, conventional concrete is only used to fill the holes below the base of the wall as a structural toe. Excavation occurs from the top down, and lagging members are placed between the installed H-piles as the excavation progresses. Lagging would likely consist of treated timber (typically 4 x 12 timber beams) and would extend on the order of 2 to 3 feet below the adjacent exposed surface of the downslope face.

Based on our subsurface explorations, we anticipate loose to medium dense fill and colluvial soils to extend to a maximum depth of 17.5 feet below the proposed roadway surface, in the vicinity of the proposed wall. Given past instability observed across the site, we anticipate that further slope instability may develop in this material over the design life of the bridge. Therefore, the soldier pile wall should be designed assuming that soils in front of the wall, extending to a depth of 17.5 feet below the roadway surface, may move away during the design life of the wall.

As indicated in Section 4.4, our stability analysis suggests that the soldier pile elements will need to extend to a minimum depth of 50 feet below the proposed roadway surface in order to provide the desired slope stabilization. Therefore, regardless of the depth required to achieve soldier pile fixity, the vertical elements will need to extend to a depth of at least 50 feet below the proposed roadway surface.

Earth Pressure Considerations

The soldier pile wall should be designed to resist lateral earth pressures shown in Figure 11. These pressure diagrams assume a maximum of one row of tieback anchors. It is assumed that the proposed wall will be free to deflect under static loading conditions and, therefore, active lateral earth pressure conditions should be assumed. Active earth pressure conditions are also assumed under seismic loading conditions. Passive earth resistance pressures are assumed to act over two shaft diameters for Strength and Service Limit State and Extreme (Seismic) Limit State. Active earth pressures are assumed to act over one shaft diameter, below the base of the excavation, and across the pile spacing for the retained portion of the wall and portions of the wall within the upper 17.5 feet below the proposed roadway grade. A resistance factor ϕ of 0.75 should be applied to the passive earth pressures for Strength and Service Limit State Design. For Extreme (Seismic) Limit State Design, a resistance factor ϕ of 1.0 should be applied to the passive earth pressures.

Because of the presence of a roadway above the slope, a traffic surcharge load should be applied to the entire length of the wall.

Tiebacks

We expect that retaining a design height of 17.5 feet will require the use of at least one row of tieback anchors. Tiebacks are installed in more-or-less horizontal rows as the excavation is stepped downward. The rows can be constructed roughly parallel to the top of the wall. Tiebacks are typically installed by cutting into the slope to form a bench at the appropriate level to work from. The tiebacks should be angled downward 20 to 30 degrees below horizontal. Although the full length of the tieback is grouted, a bond breaker such as a grease coating protected by plastic sheathing is used in the no-load zone. This forces the tieback to develop its capacity in the soils behind the no-load zone. Our recommended geometry for the no-load-zone is shown in Figure 11.

For design purposes, we recommend the ultimate pullout capacity of the tieback, referred to as a factored design load (FDL), be estimated based on 2,000 pounds per square foot of anchor surface area.

Due to the presence of dilatant clay soils, we recommend that verification testing be completed on site. Verification anchors should be installed prior to beginning installation of the production anchors. The verification anchors should be load tested to at least 150 percent of the tieback's factored design load and held for at least one hour to verify the anchor design, installation methods, equipment, and materials.

The first production anchor shall be performance tested. A minimum of 5 percent of the wall anchors, or a minimum of 3 anchors should be tested for performance, whichever is greater. The

performance anchors should be load tested to at least 100 percent of the tieback's factored design load and loaded with a load-unload process using the appropriate installation methods, equipment, and materials. The performance test shall be made by incrementally loading and unloading the ground anchor in accordance with Section 6-17.3(8)B of the WSDOT *Standard Specifications* (WSDOT, 2021)

Each production tieback should be proof loaded to at least 100 percent of its factored design load, held for at least 10 minutes. Upon completion of the test, the load shall be adjusted to the lock-off load, estimated as 80 percent of the FDL, and transferred to the anchorage device. All anchors should be proof tested as indicated in Section 6-17.3(8)C of the WSDOT *Standard Specifications* (WSDOT, 2021).

Actual tieback design, including grout mix design, anchor length, tendon design, and drilling and grouting methods should be designed by the contractor. The contractor should then be responsible for achieving the design capacity of each anchor. However, we recommend that an expanding fluidizing agent be used to mitigate grout shrinkage when installing anchors.

Portions of the soldier pile wall, close to the abutment wall, are expected to require fill placement behind the wall after installation. Sufficient fill will need to be placed prior to any tieback installation, to provide a media to drill the anchors through.

Wall Drainage Recommendations

Adequate drainage behind the soldier pile wall is critical for long term performance. We recommend prefabricated geosynthetic drain panels meeting the requirements of the WSDOT Special Provisions be placed on the wood lagging before casting the permanent fascia and tight lined into the drainage at the base of the wall. Drainage at the base of the wall should consist of a minimum 6-inch diameter perforated pipe, surrounded in free-draining material meeting the requirements of Section 9-03.12(4) Gravel Backfill for Drains of the WSDOT *Standard Specifications* (WSDOT, 2021). The drain rock should be wrapped in geotextile filter fabric meeting the requirements of Section 9-33.2(1) Tables 1 and 2 of the WSDOT *Standard Specifications* (WSDOT, 2021). The drain should be sloped to a storm drain system or another appropriate outlet. It is likely that the elevation of the wall drain will be such that gravity flows will need to be piped to the base of the ravine and outfall at a stabilized location. We recommend that any conveyance pipes, anchored to the surface of the ravine slope, consist of fused HDPE or equivalent material such that separation of the tightline pipe does not occur in the event of near surface slope movement.

General Wall Construction Considerations

Subsurface conditions encountered to the extent of our exploration consisted of loose to medium dense fill soils and soft to medium stiff colluvium. Ground water was not encountered in our borings at the bridge abutments but is anticipated at depths greater than the extent of the proposed soldier pile wall. However, perched groundwater conditions may develop which could

saturate these near surface soils, making them susceptible to caving during drilling for the soldier pile elements. This potential will increase during the wet season. The contract documents should require the contractor to assume that the use of temporary casing will be required from the proposed roadway grade to the design embedment depth of the soldier pile elements.

We expect that ground water seepage into the shaft excavations through water bearing sand seams may occur and standing water may be present at the base of the excavations prior to placement of the soldier pile elements and concrete. To facilitate displacement of the standing water, concrete should be pumped to the base of the excavation rather than end-dumped from the surface. While not encountered in our explorations, prospective contractors should also anticipate and make allowance for potential obstructions within the glacial soils during advancement of the shaft excavations.

Several active and abandoned pipes are observed extending out of the slope face, near the top of slope. We recommend that the design team locate and inventory these subsurface pipes. Soldier pile elements should be positioned to avoid hitting any of these pipes.

4.8.2 Southwestern (LaMar Drive) Retaining Wall

It is our understanding that the improvements to the existing LaMar Drive will require roadway widening and relocation further into the existing hillslope to the south. This will require a cut wall with retained heights up to 10 feet to widen the roadway for the proposed improvements. We recommend this wall be constructed as a soldier pile and lagging wall to limit the temporary excavation that would be needed for construction of structural earth walls or gravity block walls at this location. Based on our subsurface explorations, we anticipate loose to medium dense fill/colluvial soils to extend to a maximum depth of 7.5 feet below the top of the proposed retaining wall.

Soldier pile wall and lagging systems rely on embedment below the retained portion of the wall to support the lateral earth pressures exerted by the retained soil. For soldier pile and lagging wall systems, steel H-piles are generally placed in drilled shafts, spaced at approximately 6- to 8-foot centers. The diameter of typical soldier pile shaft excavations is on the order of 2 to 3 feet and the H-piles are imbedded below the bottom of the excavations. Once the H-piles are installed, the drilled shafts are filled with concrete or controlled density flowable fill (CDF). Typically, conventional concrete is only used to fill the holes below the base of the wall as a structural toe. Excavation occurs from the top down, and lagging members are placed between the installed H-piles as the excavation progresses. Lagging would likely consist of treated timber (typically 4 x 12 timber beams) and would extend on the order of 2 to 3 feet below the adjacent exposed surface of the downslope face.

Soldier Pile Wall Design Parameters

The soldier pile wall should be designed to resist lateral earth pressures shown in Figure 12. These pressure diagrams assume that no tieback anchors will be required. It is assumed that the proposed wall will be free to deflect under static loading conditions and, therefore, active lateral earth pressure conditions should be assumed. Active earth pressure conditions are also assumed under seismic loading conditions. Passive earth resistance pressures are assumed to act over two shaft diameters for Strength and Service Limit State and Extreme (Seismic) Limit State. Active earth pressures are assumed to act over one shaft diameter, below the base of the excavation, and over the width of the lagging for the retained portion of the wall. A resistance factor ϕ of 0.75 should be applied to the passive earth pressures for Strength and Service Limit State Design. For Extreme (Seismic) Limit State Design, a resistance factor ϕ of 1.0 should be applied to the passive earth pressures. The passive pressure provided assumes the pavement section will be constructed at the front of the proposed wall with marginal slope and no backslope or sustained traffic surcharge behind the proposed wall.

Wall Drainage Recommendations

Adequate drainage behind the soldier pile wall is critical for long term performance. We recommend prefabricated geosynthetic drain panels meeting the requirements of the WSDOT Special Provisions be placed on the wood lagging before casting the permanent fascia and tight lined into the drainage at the base of the wall. Drainage at the base of the wall should consist of a minimum 6-inch diameter perforated pipe, surrounded in free-draining material meeting the requirements of Section 9-03.12(4) Gravel Backfill for Drains of the WSDOT *Standard Specifications* (WSDOT, 2021). The drain rock should be wrapped in geotextile filter fabric meeting the requirements of Section 9-33.2(1) Tables 1 and 2 of the WSDOT *Standard Specifications* (WSDOT, 2021). The drain should be sloped to a storm drain system or another appropriate outlet. We recommend that any conveyance pipes consist of fused HDPE or equivalent material such that separation of the tightline pipe does not occur in the event of near surface slope movement.

Soldier Pile Wall Construction

The very loose to medium dense fill encountered in the upper 5 to 7.5 feet at this location may experience caving. Ground water was not encountered in our borings at the soldier pile wall but is anticipated at depths greater than the extent of the proposed soldier pile wall. However, perched groundwater conditions may develop which could saturate these near surface soils, making them susceptible to caving during drilling for the soldier pile elements. This potential will increase during the wet season. The contract documents should require the contractor to assume that the use of temporary casing will be required from the proposed roadway grade to the design embedment depth of the soldier pile elements.

Below the near surface fill soils, the excavations will be advanced through stiff to hard transitional bed deposits underlain by very dense Whidbey formation sands; hard drilling

conditions may be encountered and should be the contractor should plan accordingly. Although not encountered in our borings, large cobbles and boulders are known to exist in these deposits. The shaft contractor should be prepared to handle cobbles and boulders, if encountered.

We expect that ground water seepage into the shaft excavations through water bearing sand seams may occur and standing water may be present at the base of the excavations prior to placement of the soldier pile elements and concrete. To facilitate displacement of the standing water, concrete should be pumped to the base of the excavation rather than end-dumped from the surface. While not encountered in our explorations, prospective contractors should also anticipate and make allowance for potential obstructions within the glacial soils during advancement of the shaft excavations.

Several active and abandoned pipes are observed extending out of the slope face within the vicinity of the proposed retaining wall. We recommend that the design team locate and inventory these subsurface pipes. Soldier pile elements should be positioned to avoid hitting any of these pipes.

4.9   STORMWATER MANAGEMENT

The project alignment was found to be underlain by fine-grained colluvial and glacial soils near the ground surface. These fine-grained deposits are considered not suitable for the use of onsite infiltration as a means of stormwater management. Additionally, on-site infiltration into near surface soils may result in an increase in soil saturation which can develop further slope instability. Therefore, we do not recommend the use of onsite infiltration as a means of stormwater management for this project. As a result, we anticipate that stormwater detention will be required as a means of stormwater management.

4.10 PAVEMENT RECOMMENDATIONS

It is our understanding that portions of the roadway, within the vicinity of the bridge approaches, will be reconstructed as part of bridge reconstruction. We recommend that the proposed pavement section be constructed using Hot Mix Asphalt (HMA). Additionally, approach slabs are recommended at each abutment to connect the pavement section to the proposed bridge structure. We anticipate these approach slabs will be constructed of Portland Cement Concrete (PCC) and designed by others. The following sections present our design recommendations for new HMA pavement.

4.10.1 Design Traffic Parameters

Current design traffic parameters were provided by the City of Everett and Gibson Traffic Consultants for the Edgewater Creek Bridge along W Mukilteo Blvd in the eastbound and westbound directions.

The traffic volumes provided were separated into the 13 FHWA Vehicle Classifications. For design, we assumed 0.008 Equivalent Single Axle Loads (ESALs) per vehicle for Classes 1-3, 3 ESALs per vehicle for Class 4 (buses), and 1.4 ESALs per vehicle for Classes 5 through 13. We also used a 4.0% annual growth rate on traffic volume, based on information provided by Gibson Traffic Consultants. A 30-year pavement design life was assumed. This resulted in the following total ESAL values:

- Eastbound: 1,282,534 ESALs

- Westbound: 1,071,660 ESALs

A value of 1,300,000 ESALs was used for pavement design.

The pavement recommendations presented in this report are based on these traffic calculations. If additional traffic count information is obtained that varies appreciably from these values, the recommendations given in this report should be reviewed and revised as necessary.

4.10.2 **New HMA Pavement Design**

Table 18 provides our new HMA design recommendations, assuming the traffic loading described above. This pavement design is based on the design method presented in the 1993 AASHTO Design Guide (AASHTO, 1993) using the following parameters:

- Reliability = 95%
- Initial Serviceability = 4.5
- Terminal Serviceability = 2.5
- Overall Standard Deviation = 0.50
- Subgrade Resilient Modulus = 10 ksi

These values result in a required AASHTO Structural Number (SN) of 3.74.

Table 18. Structure Requirements for New HMA Pavement – 30-Year Design Life

Material Description	Option A Minimum Layer Thickness (inches)	Option B Minimum Layer Thickness (inches)	WSDOT Standard Specification
HMA	7	6	5-04 & 9-02.1
CSBC	5	8	9-03.9(3)

We recommend that the asphaltic layers consist of HMA Class 1/2-inch. The upper wearing course (2-3 inches) could consist of HMA Class 3/8-inch. Recommendations are presented below for subgrade preparation and structural fill placement and compaction for pavement reconstruction.

The pavement will likely require periodic maintenance. Cracks larger than 1/4-inch in width should be sealed periodically. The pavement will likely require a functional overlay after about 10 to 12 years because of non-structural distresses caused by environmental factors such as degradation of the asphalt surface.

4.10.3 **HMA Binder Selection**

The selection of the optimum asphalt binder type for the prevailing climate is critical to ensure long-term pavement performance. Use of the wrong binder can result in low temperature cracking or permanent deformation at high temperatures.

Based on the climate in Everett, and the traffic volumes provided, we recommend Superpave Performance Grade binder PG 58H-22 be used for new pavements.

4.10.4 **Placement of HMA**

Placement of HMA should be in accordance with Section 5-04 of the WSDOT *Standard Specifications* (WSDOT, 2021). Particular attention should be paid to the following:

- HMA should not be placed until the engineer has accepted the previously constructed pavement layers.

- HMA should not be placed on any frozen or wet surface.

- HMA should not be placed when precipitation is anticipated before the pavement can be compacted, or before any other weather conditions which could prevent proper handling and compaction of HMA.

- HMA should not be placed when the average surface temperatures are less than 45 °F.

- HMA temperature behind the paver should be in excess of 240 °F. Compaction should be completed before the mix temperature drops below 180 °F. Comprehensive temperature records should be kept during the HMA placement.

- Sufficient tack coat must be applied uniformly and allowed to break and set before placing HMA above an existing HMA layer in order to create a strong bond between layers. The surface of the pavement should be thoroughly cleaned prior to tack coat application. Improper tack coat application can cause unbonded layers and will lead to premature pavement distress/failure.

- For cold joints, tack coat should be applied to the edge to be joined and the paver screed should be set to overlap the first mat by 1 to 2 inches.

4.10.5 Pavement Subgrade Preparation

Site preparation for pavement reconstruction should begin with the excavation of all existing materials down to a depth sufficient to accommodate the new structure. Based on the results of the subsurface explorations, silty sand fill soils are anticipated at the base of the excavation. The exposed soils should be thoroughly compacted and evaluated by a geotechnical engineer or qualified earthworks inspector. If loose, pumping, or otherwise unsuitable soils are encountered at the bottom of the pavement section, they should be over-excavated as directed by the geotechnical engineer and backfilled using Crushed Surfacing Base Course (CSBC) per the recommendations in the following section.

4.10.6 Pavement Structural Fill and Compaction

Imported structural fill for pavement sub-base and base course should consist of Crushed Surfacing Base Course (CSBC), as described in Section 9-03.9(3) of the *Standard Specifications* (WSDOT, 2021).

Structural fill should be placed in loose, horizontal, lifts of not more than 8 inches in thickness and compacted to at least 95% of the maximum dry density, as determined using test method ASTM D 1557 (Modified Proctor). At the time of placement, the moisture content of structural fill should be at or near optimum. The procedure required to achieve the specified minimum relative compaction depends on the size and type of compaction equipment, the number of passes, thickness of the layer being compacted, and the soil moisture-density properties.

When the first fill is placed in a given area, and/or anytime the fill material changes, the area should be considered a test section. The test section should be used to establish fill placement and compaction procedures required to achieve proper compaction. The geotechnical consultant should observe placement and compaction of the test section to assist in establishing an appropriate compaction procedure. Once a placement and compaction procedure are established, the contractor's operations should be monitored, and periodic density tests performed to verify that proper compaction is being achieved.

Generally, loosely compacted soils result from poor construction technique or improper moisture content. Soils with a high percentage of silt or clay content are particularly susceptible to becoming too wet, and coarse-grained materials easily become too dry, for proper compaction. Silty or clayey soils with a moisture content too high for adequate compaction should be dried, as necessary, or moisture conditioned by mixing with drier materials, or other treatment methods. For coarse-grained structural fill soils, moisture conditioning by sprinkling before and during compaction is sometimes required to achieve the required relative compaction.

4.10.7 **Pavement Drainage**

It is essential to the satisfactory performance of the roadway that good drainage is provided to prevent water ponding on or alongside, or accumulating beneath, the pavement. Water ponding can cause saturation of the pavement and subgrade layers and lead to premature failure. The base layers and subgrade surface should be graded to prevent water being trapped within the layer. The surface of the pavement should be sloped to convey water from the pavement to appropriate drainage facilities.

5. EARTHWORK

5.1 STRUCTURAL FILL

Materials used as backfill for the project are considered "Structural Fill". The onsite soils are highly variable in composition and moisture sensitive. We do not recommend reusing the onsite soils as structural fill for this project. Structural fill should consist of imported clean, free-draining, granular soils free from organic matter or other deleterious materials. Such materials should be less than 4 inches in maximum particle dimension, with less than 7 percent fines (portion passing the U.S. Standard No. 200 sieve), as specified for Gravel Borrow in Section 9-03.14(1) of the 2021 WSDOT *Standard Specifications*. The fine-grained portion of structural fill soils should be non-plastic. Backfill within the reinforced zone of wing walls should consist of Gravel Borrow for Structural Earth Walls, as described in Section 9-03.14(4) of the *Standard Specifications* (WSDOT, 2021).

5.2 COMPACTION

Structural fill soils should be moisture conditioned and compacted to the requirements specified in Section 2-03.3(14), Method C, of the 2021 WSDOT *Standard Specifications*, except that maximum dry densities should be obtained using ASTM D 1557 (Modified Proctor). Achievement of proper density of a compacted fill depends on the size and type of compaction equipment, the number of passes, thickness of the layer being compacted, and soil moisture-density properties. In areas where limited space restricts the use of heavy equipment, smaller equipment can be used, but the soil must be placed in thin enough layers to achieve the required relative compaction.

In order to minimize subsequent settlement of the excavation backfill and new pavements, we recommended that backfill soils be placed in loose, lifts no thicker than 8 inches and each lift should be compacted to at least 95 percent of its Modified Proctor maximum density (ASTM D 1557). The procedure to achieve proper density of compacted fill depends on the size and type of compaction equipment, the number of passes, thickness of the layer being compacted, and soil moisture-density properties.

5.3 TEMPORARY EXCAVATION

We anticipate that the bridge approaches and associated wing walls may require the installation of temporary excavation slopes. Maintenance of safe working conditions, including temporary excavation stability, is the responsibility of the contractor. All temporary excavations more than 4 feet in depth must be sloped in accordance with Part N of WAC (Washington Administrative Code) 296-155 or be shored. The fill, colluvium, and transitional bed soils will classify as Type C soil, for which WAC requires that unsupported excavation must be inclined no steeper than 1.5H:1V. This assumes that adequate dewatering has been provided to maintain stable slopes during excavation. Flatter slopes may be necessary where near surface runoff or ground water impacts the stability of the temporary slopes. The slopes should be monitored, and slope angles adjusted in the field based on local subsurface conditions and the contractor's methods.

The design, installation, maintenance and removal of temporary shoring should be the responsibility of the contractor. The shoring system should be designed by a qualified and licensed engineer experienced with shoring design for deep excavations within similar soil conditions. We recommend that the design of the temporary shoring system be submitted by the contractor, for approval, prior to starting excavation. HWA should be allowed to review shop drawings and calculations for proposed shoring systems to check for consistency with the recommendations included in this report.

5.4 WET WEATHER EARTHWORK

General recommendations relative to earthwork performed in wet weather or in wet conditions are presented below. These recommendations should be incorporated into the contract specifications.

- Earthwork should be performed in small areas to minimize exposure to wet weather. Excavation of unsuitable and/or softened soil should be followed promptly by placement and compaction of clean structural fill. The size and type of construction equipment used may need to be limited to prevent soil disturbance. Under some circumstances, it may be necessary to excavate soils with a backhoe to minimize subgrade disturbance caused by equipment traffic.

- Material used as excavation backfill in wet weather should consist of clean granular soil with less than 5 percent passing the U.S. No. 200 sieve, based on wet sieving the fraction passing the ¾-inch sieve. The fines should be non-plastic. It should be noted this is an additional restriction on the structural fill materials specified.

- The ground surface within the construction area should be graded to promote surface water run-off and to prevent ponding.

- Within the construction area, the ground surface should be sealed on completion of each shift by a smooth drum vibratory roller, or equivalent, and under no circumstances should soil be left uncompacted and exposed to moisture infiltration.

- Excavation and placement of backfill materials should be monitored by a geotechnical engineer experienced in wet weather earthwork to determine that the work is being accomplished in accordance with the project specifications and the recommendations contained herein.

6. CONDITIONS AND LIMITATIONS

We have prepared this report for the City of Everett and the TranTech design team for use in evaluation of this project. The conclusions and interpretations presented in this report should not be construed as our warranty of subsurface conditions at the site. Experience has shown that soil and ground water conditions can vary significantly over small distances and with time. Inconsistent conditions can occur between explorations that may not be detected by a geotechnical study of this scope and nature.

Within the limitations of approved scope, schedule and budget, HWA attempted to execute these services in accordance with generally accepted professional principles and practices in the fields of geotechnical engineering and engineering geology at the time the report was prepared. No warranty, express or implied, is made.

HWA does not practice or consult in the field of safety engineering. We do not direct the contractor's operations and cannot be responsible for the safety of personnel other than our own on the site. As such, the safety of others is the responsibility of the contractor. However, the contractor should notify the owner if any of the recommended actions presented herein are considered unsafe.

We appreciate the opportunity to provide geotechnical services on this project. Should you have any questions or comments, or if we may be of further service, please do not hesitate to call.

Sincerely,

HWA GEOSCIENCES INC.

Sean Schlitt, P.E.   Donald Huling, P.E.
Geotechnical Engineer   Geotechnical Engineer, Principal

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SITE MAP

0   200'   400'   600'   800'

N

SCALE: 1" = 400'

VICINITY MAP

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0   1000'   2000'   3000'   4000'

N

SCALE: 1" = 2000'
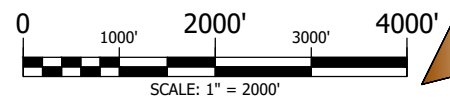
HWA GeoSciences Inc. DBE/MWBE	SITE AND VICINITY MAP
	EDGEWATER BRIDGE REPLACEMENT EVERETT, WASHINGTON

FIGURE NO.:

1

DRAWN BY: CF   CHECK BY: SKS

PROJECT #
2019-157-21

EDGEWATER BRIDGE REPLACEMENT
Scale: 1" = 60'-0"
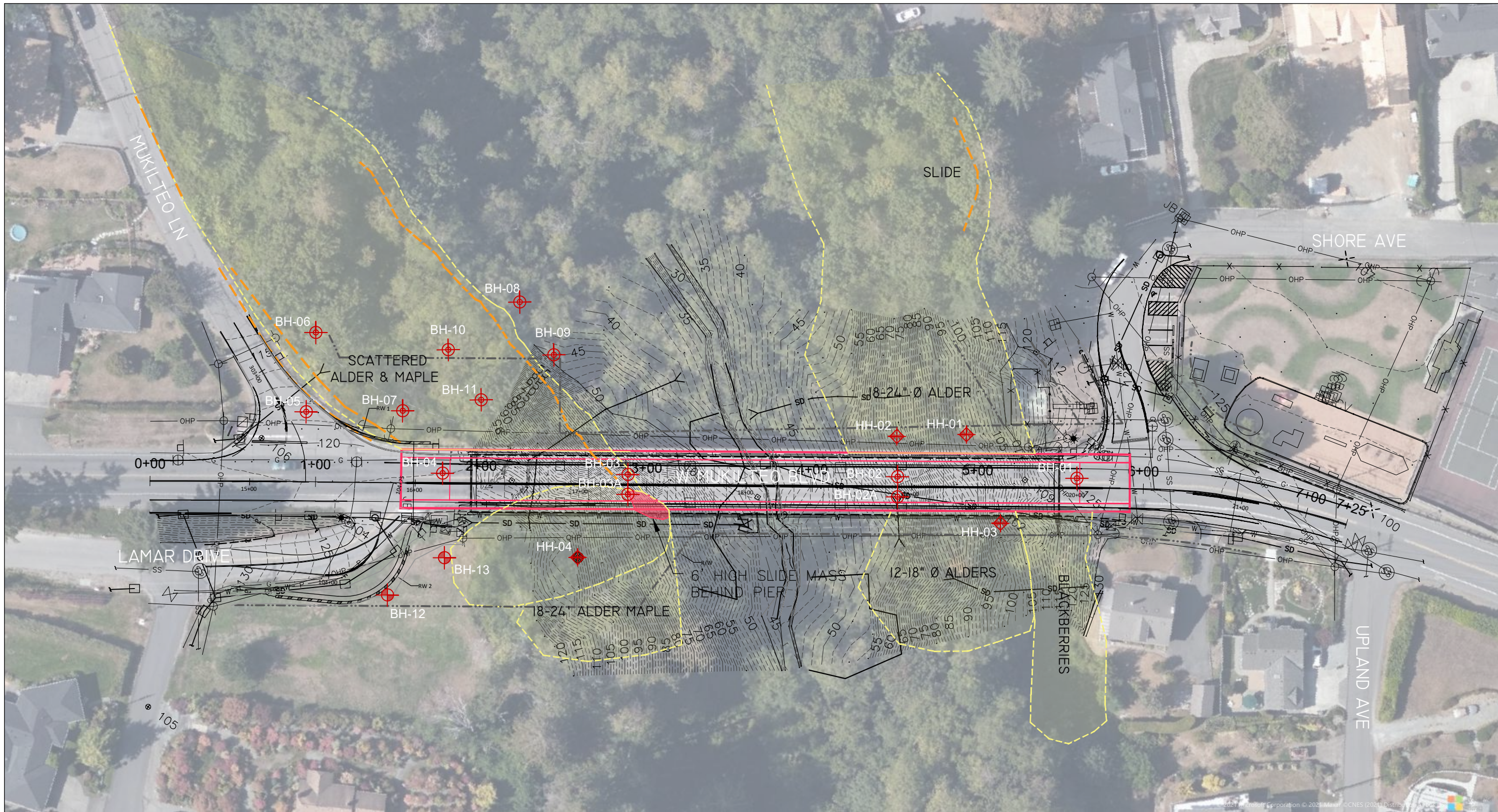
EXPLORATION LEGEND

BH-12 ⊕ BOREHOLE DESIGNATION AND APPROXIMATE LOCATION (HWA, 2021)

BH-07 ⊕ BOREHOLE DESIGNATION AND APPROXIMATE LOCATION (HWA, 2020)

HH-04 ◈ HANDHOLE DESIGNATION AND APPROXIMATE LOCATION (HWA, 2020)

– – – PROPOSED WALL

⋯⋯ PREVIOUS ROW

‒ ‒ ‒ SCARPS AND INCIPIENT SCARPS

▨ SLIDE AREA BY BRIDGE

▨ SLIDE DEBRIS

⋯⋯ PROPOSED ROW

BASE MAP PROVIDED BY: BING AND SURVEYOR
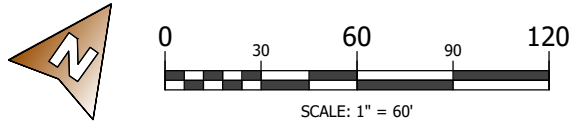
HWA GeoSciences Inc.
DBE/MWBE

GEOTECHNICAL EXPLORATION PLAN
EDGEWATER BRIDGE REPLACEMENT
EVERETT, WASHINGTON

SITE & EXPLORATION PLAN

DRAWN BY: CF

CHECK BY: SKS/MAB

FIGURE NO.: 2

PROJECT NO.: 2019-157-21

SCALE: 1" = 60'

0   30   60   90   120

EDGEWATER BRIDGE REPLACEMENT

BORE LEGEND

BH-3 — EXPLORATION DESIGNATION

8 / 15 / 20 / 26 / 28 / 30 — BLOW COUNT "N-VALUE"

▽ — WATER LEVEL IN WELL AND DATE

59 / 54 / 25 / 35 — INFERRED GEOLOGIC CONTACT

46.5 — BOTTOM OF EXPLORATION
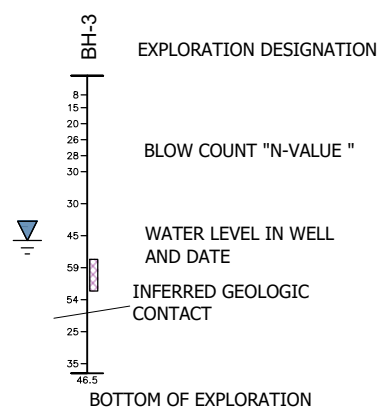
SOILS LEGEND

FILL

COLLUVIUM (FINE-GRAINED)

TRANSITIONAL BEDS

WHIDBEY FORMATION

POTENTIAL EXTENT OF LIQUEFACTION SUSCEPTIBLE SOILS

BASE MAP PROVIDED BY: BING AND SURVEYOR

C:\USERS\CFRY\DESKTOP\VARIOUS INFO FOLDER\2019-157-21 EDGEWATER BRIDGE\2019-157-21 EDGEWATER BRIDGE_CROSS SECTION.DWG <3> Plotted: 11/3/2020 2:48 PM

HWA GeoSciences Inc.
DBE/MWBE
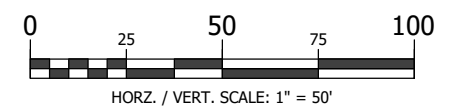
EDGEWATER BRIDGE REPLACEMENT
EVERETT, WASHINGTON

GEOLOGIC CROSS
SECTION A-A'

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CF	**3**
CHECK BY:	PROJECT NO.:
SKS	2019-157-21
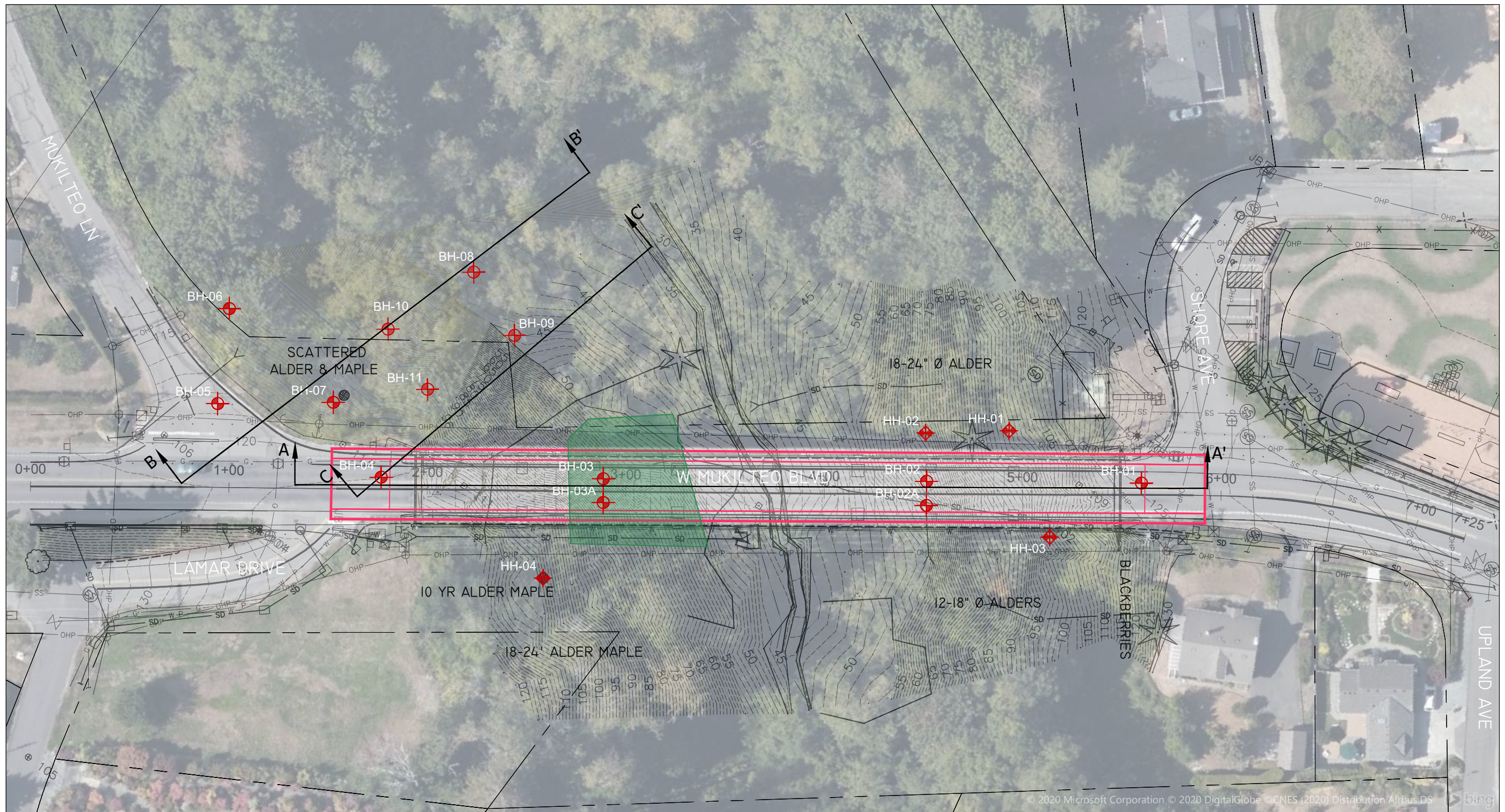
EXPLORATION LEGEND

BH-07 ⊕ BOREHOLE DESIGNATION AND APPROXIMATE LOCATION (HWA, 2020)

HH-04 ◆ HANDHOLE DESIGNATION AND APPROXIMATE LOCATION (HWA, 2020)

AREA OF PROPOSED EXCAVATION

EDGEWATER BRIDGE REPLACEMENT
Scale: 1" = 50'-0"

GEOLOGIC PROFILE A-A' - CENTERLINE OF THE PROPOSED BRIDGE
SLOPE STABILITY PROFILE B-B' - NORTH OF THE WING WALLS
SLOPE STABILITY PROFILE C-C' - CRITICAL SLOPE

A ↑   ↑ A'
         GEOLOGIC CROSS SECTION

BASE MAP PROVIDED BY: BING AND SURVEYOR

SCALE: 1" = 50'

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EDGEWATER BRIDGE REPLACEMENT
EVERETT, WASHINGTON

PLAN VIEW OF AREA
OF EXCAVATION

DRAWN BY: CF

CHECK BY: SKS

FIGURE NO.: 4

PROJECT NO.: 2019-157-21

C:\USERS\CFRY\DESKTOP\VARIOUS INFO FOLDER\2019-157-21 EDGEWATER BRIDGE\2019-157-21 EDGEWATER BRIDGE_10.06.2020.DWG <4> Plotted: 11/3/2020 2:08 PM

CROSS SECTION OF AREA OF EXCAVATION

EDGEWATER BRIDGE REPLACEMENT
EVERETT, WASHINGTON

FIGURE NO.:

5

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PROJECT #
2019-157-21

HWA GeoSciences Inc.
DBE/MWBE
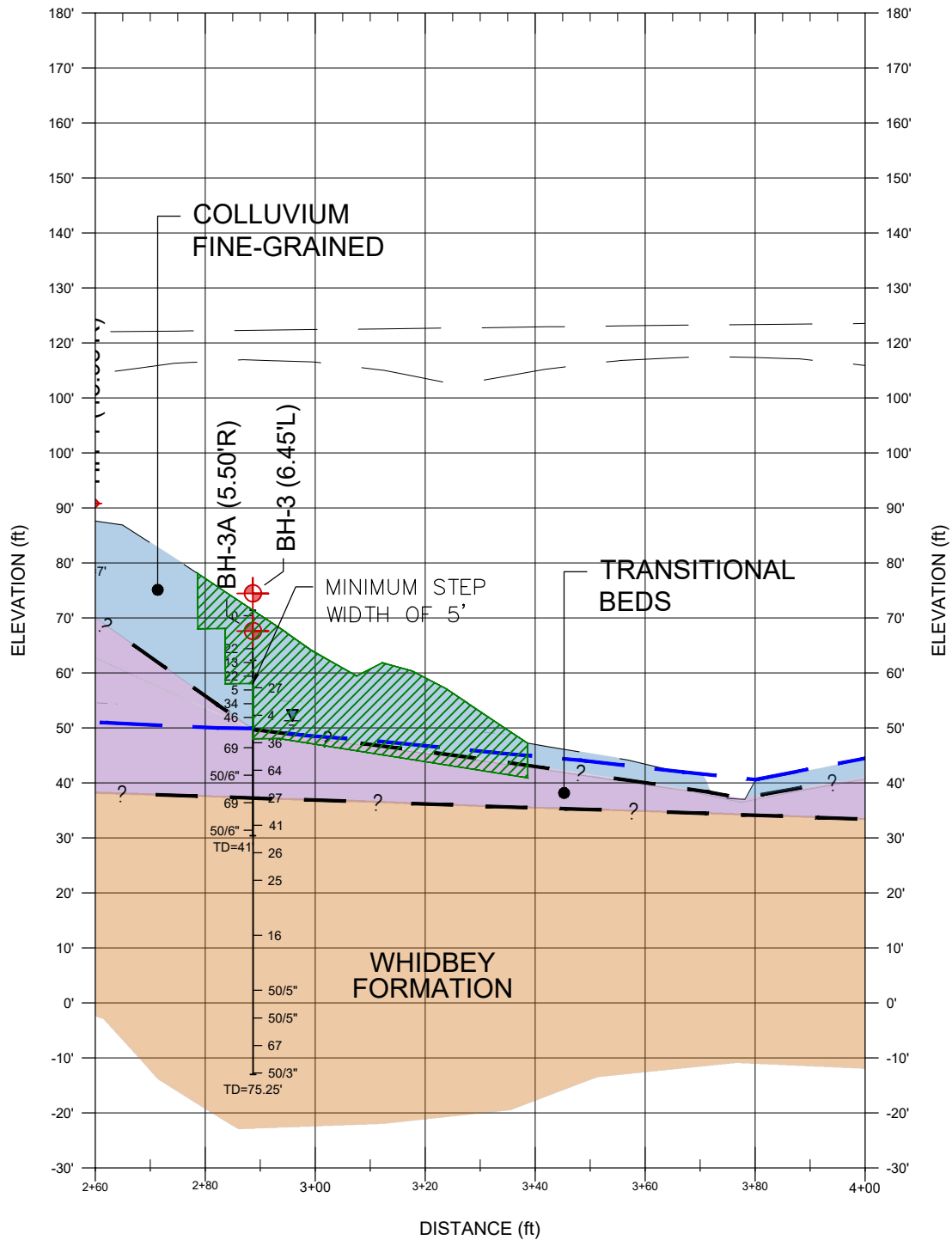
LEGEND

AREA OF EXCAVATION

SCALE: 1" = 30'

ASSUMED SUBSURFACE PROFILE

ELEVATION (feet)

Elevation	Layer
125' – 107.5'	FILL
107.5' – 45'	TRANSITIONAL BEDS
45' – 24'	WHIDBEY FORMATION

SERVICE LIMIT

AXIAL RESISTANCE (kips)

Legend:
- Nominal Side: 1-inch Settlement
- Nominal Base: 1-inch Settlement
- Factored Total: 1-inch Settlement

SERVICE LIMIT NOTES:

1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification is 1.0 for side resistance.

2. Settlements is based on a single shaft. No group action is considered.

3. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

STRENGTH LIMIT

AXIAL RESISTANCE (kips)

Legend:
- Nominal Side
- Nominal Base
- Factored Total

STRENGTH LIMIT NOTES:

1. Recommended resistance factors included in Factored Loads are 0.55 for cohesionless and 0.45 for cohesive for side resistance and 0.5 for cohesionless and 0.4 for cohesive for base resistance, as provided in AASHTO LRFD Bridge Design Specification.

2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.35 (per AASHTO LRFD Bridge Design Specification).

3. Recommended load factor of 1.25 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

EXTREME EVENT LIMIT

AXIAL RESISTANCE (kips)

Legend:
- Nominal Side
- Nominal Base
- Factored Total

EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification for both side and base resistance are 1.0 for compression and 0.8 for uplift.

2. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

GENERAL NOTES:

1. The analyses were performed based on guidelines included in the AASHTO LRFD Bridge Design Specification and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts.

2. Factored total shaft resistance shown on plots include the summation of the shaft's nominal side and base resistances multiplied by the appropriate resistance factors as noted above.

3. The nominal side and base resistance values presented do not include the resistance factors.

4. The nominal base and total factored axial capacities provided have been are reduced to account for the weight of drilled shafts with appropriate load factors applied for each limit state.

5. The weight of the drilled shafts is calculated from the proposed shaft top elevation presented on the assumed subsurface profile.

HWA GeoSciences Inc.

EDGEWATER CREEK
BRIDGE REPLACEMENT
EVERETT, WASHINGTON

EASTERN ABUTMENT (BH-1)
AXIAL SHAFT CAPACITIES
1.5-METER DIAMETER SHAFT

DRAWN BY	SKS	FIGURE NO.
CHECKED BY	DJH	**6A**
DATE	05.07.20	PROJECT NO. 2019-157-21

ASSUMED SUBSURFACE PROFILE

Subsurface profile (ELEVATION in feet):
- 64' / 62.5' — FILL
- COLLUVIUM
- 54' — TRANSITIONAL BEDS
- 31' — WHIDBEY FORMATION
- -12'

SERVICE LIMIT

AXIAL RESISTANCE (kips)

Legend:
- Nominal Side: 1-inch Settlement
- Nominal Base: 1-inch Settlement
- Factored Total: 1-inch Settlement

SERVICE LIMIT NOTES:

1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification is 1.0 for side resistance.

2. Settlements is based on a single shaft. No group action is considered.

3. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

STRENGTH LIMIT

AXIAL RESISTANCE (kips)

Legend:
- Nominal Side
- Nominal Base
- Factored Total

STRENGTH LIMIT NOTES:

1. Recommended resistance factors included in Factored Loads are 0.55 for cohesionless and 0.45 for cohesive for side resistance and 0.5 for cohesionless and 0.4 for cohesive for base resistance, as provided in AASHTO LRFD Bridge Design Specification.

2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.35 (per AASHTO LRFD Bridge Design Specification).

3. Recommended load factor of 1.25 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

EXTREME EVENT LIMIT

AXIAL RESISTANCE (kips)

Legend:
- Nominal Side
- Nominal Base
- Factored Total

EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification for both side and base resistance are 1.0 for compression and 0.8 for uplift.

2. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

GENERAL NOTES:

1. The analyses were performed based on guidelines included in the AASHTO LRFD Bridge Design Specification and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts.

2. Factored total shaft resistance shown on plots include the summation of the shaft's nominal side and base resistances multiplied by the appropriate resistance factors as noted above.

3. The nominal side and base resistance values presented do not include the resistance factors.

4. The nominal base and total factored axial capacities provided have been are reduced to account for the weight of drilled shafts with appropriate load factors applied for each limit state.

5. The weight of the drilled shafts is calculated from the proposed shaft top elevation presented on the assumed subsurface profile.

HWA GEOSCIENCES INC.	EDGEWATER CREEK BRIDGE REPLACEMENT EVERETT, WASHINGTON

EASTERN INTERIOR PIER (BH-2A)
AXIAL SHAFT CAPACITIES
1.5-METER DIAMETER SHAFT

DRAWN BY: SKS
CHECKED BY: DJH
DATE: 05.07.20

FIGURE NO. **6B**

PROJECT NO. 2019-157-21

ASSUMED SUBSURFACE PROFILE

ELEVATION (feet)

- 62' — COLLUVIUM
- 50' — TRANSITIONAL BEDS
- 37' — WHIDBEY FORMATION
- -12.25'

SERVICE LIMIT

AXIAL RESISTANCE (kips)

Legend:
- Nominal Side: 1-inch Settlement
- Nominal Base: 1-inch Settlement
- Factored Total: 1-inch Settlement

STRENGTH LIMIT

AXIAL RESISTANCE (kips)

Legend:
- Nominal Side
- Nominal Base
- Factored Total

EXTREME EVENT LIMIT

AXIAL RESISTANCE (kips)

Legend:
- Nominal Side
- Nominal Base
- Factored Total

Add Downdrag Loads to Other Foundation Loads
(see Extreme Event Limit Note 2)

SERVICE LIMIT NOTES:

1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification is 1.0 for side resistance.

2. Settlements is based on a single shaft. No group action is considered.

3. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

STRENGTH LIMIT NOTES:

1. Recommended resistance factors included in Factored Loads are 0.55 for cohesionless and 0.45 for cohesive for side resistance and 0.5 for cohesionless and 0.4 for cohesive for base resistance, as provided in AASHTO LRFD Bridge Design Specification.

2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.35 (per AASHTO LRFD Bridge Design Specification).

3. Recommended load factor of 1.25 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification for both side and base resistance are 1.0 for compression and 0.8 for uplift.

2. Unfactored static downdrag force, due to liquefaction-induced settlement, for each shaft is estimated to be **165 kips.** A load factor of 1.05 should be applied to all downdrag loads (Allen, 2005) to determine factored downdrag force.

3. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

GENERAL NOTES:

1. The analyses were performed based on guidelines included in the AASHTO LRFD Bridge Design Specification and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts.

2. Factored total shaft resistance shown on plots include the summation of the shaft's nominal side and base resistances multiplied by the appropriate resistance factors as noted above.

3. The nominal side and base resistance values presented do not include the resistance factors.

4. The nominal base and total factored axial capacities provided have been are reduced to account for the weight of drilled shafts with appropriate load factors applied for each limit state.

5. The weight of the drilled shafts is calculated from the proposed shaft top elevation presented on the assumed subsurface profile.

HWA GeoSciences Inc.	EDGEWATER CREEK BRIDGE REPLACEMENT EVERETT, WASHINGTON	WESTERN INTERIOR PIER (BH-3A) AXIAL SHAFT CAPACITIES 1.5-METER DIAMETER SHAFT	DRAWN BY: SKS / CHECKED BY: DJH / DATE: 05.07.20 — FIGURE NO. **6C** — PROJECT NO. 2019-157-21

ASSUMED SUBSURFACE PROFILE

- 121' — FILL
- 111' — COLLUVIUM
- 108.5'
- TRANSITIONAL BEDS
- 41'
- WHIDBEY FORMATION
- 19.5'

ELEVATION (feet)

SERVICE LIMIT

AXIAL RESISTANCE (kips)

Legend:
- Nominal Side: 1-inch Settlement
- Nominal Base: 1-inch Settlement
- Factored Total: 1-inch Settlement

SERVICE LIMIT NOTES:

1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification is 1.0 for side resistance.

2. Settlements is based on a single shaft. No group action is considered.

3. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

STRENGTH LIMIT

AXIAL RESISTANCE (kips)

Legend:
- Nominal Side
- Nominal Base
- Factored Total

STRENGTH LIMIT NOTES:

1. Recommended resistance factors included in Factored Loads are 0.55 for cohesionless and 0.45 for cohesive for side resistance and 0.5 for cohesionless and 0.4 for cohesive for base resistance, as provided in AASHTO LRFD Bridge Design Specification.

2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.35 (per AASHTO LRFD Bridge Design Specification).

3. Recommended load factor of 1.25 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

EXTREME EVENT LIMIT

AXIAL RESISTANCE (kips)

Legend:
- Nominal Side
- Nominal Base
- Factored Total

EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification for both side and base resistance are 1.0 for compression and 0.8 for uplift.

2. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

GENERAL NOTES:

1. The analyses were performed based on guidelines included in the AASHTO LRFD Bridge Design Specification and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts.

2. Factored total shaft resistance shown on plots include the summation of the shaft's nominal side and base resistances multiplied by the appropriate resistance factors as noted above.

3. The nominal side and base resistance values presented do not include the resistance factors.

4. The nominal base and total factored axial capacities provided have been are reduced to account for the weight of drilled shafts with appropriate load factors applied for each limit state.

5. The weight of the drilled shafts is calculated from the proposed shaft top elevation presented on the assumed subsurface profile.

ASSUMED SUBSURFACE PROFILE

Subsurface profile (ELEVATION, feet):
- 125' to 107.5': FILL
- 107.5' to 45': TRANSITIONAL BEDS
- 45' to 24': WHIDBEY FORMATION

SERVICE LIMIT

AXIAL RESISTANCE (kips)

Legend:
- Nominal Side: 1-inch Settlement
- Nominal Base: 1-inch Settlement
- Factored Total: 1-inch Settlement

SERVICE LIMIT NOTES:

1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification is 1.0 for side resistance.

2. Settlements is based on a single shaft. No group action is considered.

3. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

STRENGTH LIMIT

AXIAL RESISTANCE (kips)

Legend:
- Nominal Side
- Nominal Base
- Factored Total

STRENGTH LIMIT NOTES:

1. Recommended resistance factors included in Factored Loads are 0.55 for cohesionless and 0.45 for cohesive for side resistance and 0.5 for cohesionless and 0.4 for cohesive for base resistance, as provided in AASHTO LRFD Bridge Design Specification.

2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.35 (per AASHTO LRFD Bridge Design Specification).

3. Recommended load factor of 1.25 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

EXTREME EVENT LIMIT

AXIAL RESISTANCE (kips)

Legend:
- Nominal Side
- Nominal Base
- Factored Total

EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification for both side and base resistance are 1.0 for compression and 0.8 for uplift.

2. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

GENERAL NOTES:

1. The analyses were performed based on guidelines included in the AASHTO LRFD Bridge Design Specification and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts.

2. Factored total shaft resistance shown on plots include the summation of the shaft's nominal side and base resistances multiplied by the appropriate resistance factors as noted above.

3. The nominal side and base resistance values presented do not include the resistance factors.

4. The nominal base and total factored axial capacities provided have been are reduced to account for the weight of drilled shafts with appropriate load factors applied for each limit state.

5. The weight of the drilled shafts is calculated from the proposed shaft top elevation presented on the assumed subsurface profile.

HWA GeoSciences Inc.	EDGEWATER CREEK BRIDGE REPLACEMENT EVERETT, WASHINGTON	EASTERN ABUTMENT (BH-1) AXIAL SHAFT CAPACITIES 2-METER DIAMETER SHAFT

DRAWN BY: SKS
CHECKED BY: DJH
DATE: 05.07.20
FIGURE NO. 7A
PROJECT NO. 2019-157-21

ASSUMED SUBSURFACE PROFILE

Elevation markers:
- 64'
- 62.5' — FILL
- 54' — COLLUVIUM
- 31' — TRANSITIONAL BEDS
- -12' — WHIDBEY FORMATION

ELEVATION (feet)

SERVICE LIMIT

AXIAL RESISTANCE (kips)

Legend:
- Nominal Side: 1-inch Settlement
- Nominal Base: 1-inch Settlement
- Factored Total: 1-inch Settlement

ELEVATION (feet)

SERVICE LIMIT NOTES:

1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification is 1.0 for side resistance.

2. Settlements is based on a single shaft. No group action is considered.

3. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

STRENGTH LIMIT

AXIAL RESISTANCE (kips)

Legend:
- Nominal Side
- Nominal Base
- Factored Total

ELEVATION (feet)

STRENGTH LIMIT NOTES:

1. Recommended resistance factors included in Factored Loads are 0.55 for cohesionless and 0.45 for cohesive for side resistance and 0.5 for cohesionless and 0.4 for cohesive for base resistance, as provided in AASHTO LRFD Bridge Design Specification.

2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.35 (per AASHTO LRFD Bridge Design Specification).

3. Recommended load factor of 1.25 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

EXTREME EVENT LIMIT

AXIAL RESISTANCE (kips)

Legend:
- Nominal Side
- Nominal Base
- Factored Total

ELEVATION (feet)

EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification for both side and base resistance are 1.0 for compression and 0.8 for uplift.

2. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

GENERAL NOTES:

1. The analyses were performed based on guidelines included in the AASHTO LRFD Bridge Design Specification and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts.

2. Factored total shaft resistance shown on plots include the summation of the shaft's nominal side and base resistances multiplied by the appropriate resistance factors as noted above.

3. The nominal side and base resistance values presented do not include the resistance factors.

4. The nominal base and total factored axial capacities provided have been are reduced to account for the weight of drilled shafts with appropriate load factors applied for each limit state.

5. The weight of the drilled shafts is calculated from the proposed shaft top elevation presented on the assumed subsurface profile.

ASSUMED SUBSURFACE PROFILE

ELEVATION (feet)

- 62' COLLUVIUM
- 50' TRANSITIONAL BEDS
- 37' WHIDBEY FORMATION
- -12.25'

SERVICE LIMIT

AXIAL RESISTANCE (kips)

Legend:
- Nominal Side: 1-inch Settlement
- Nominal Base: 1-inch Settlement
- Factored Total: 1-inch Settlement

STRENGTH LIMIT

AXIAL RESISTANCE (kips)

Legend:
- Nominal Side
- Nominal Base
- Factored Total

EXTREME EVENT LIMIT

AXIAL RESISTANCE (kips)

Legend:
- Nominal Side
- Nominal Base
- Factored Total

Add Downdrag Loads to Other Foundation Loads (see Extreme Event Limit Note 2)

SERVICE LIMIT NOTES:

1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification is 1.0 for side resistance.

2. Settlements is based on a single shaft. No group action is considered.

3. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

STRENGTH LIMIT NOTES:

1. Recommended resistance factors included in Factored Loads are 0.55 for cohesionless and 0.45 for cohesive for side resistance and 0.5 for cohesionless and 0.4 for cohesive for base resistance, as provided in AASHTO LRFD Bridge Design Specification.

2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.35 (per AASHTO LRFD Bridge Design Specification).

3. Recommended load factor of 1.25 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification for both side and base resistance are 1.0 for compression and 0.8 for uplift.

2. Unfactored static downdrag force, due to liquefaction-induced settlement, for each shaft is estimated to be **220 kips.** A load factor of 1.05 should be applied to all downdrag loads (Allen, 2005) to determine factored downdrag force.

3. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

GENERAL NOTES:

1. The analyses were performed based on guidelines included in the AASHTO LRFD Bridge Design Specification and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts.

2. Factored total shaft resistance shown on plots include the summation of the shaft's nominal side and base resistances multiplied by the appropriate resistance factors as noted above.

3. The nominal side and base resistance values presented do not include the resistance factors.

4. The nominal base and total factored axial capacities provided have been are reduced to account for the weight of drilled shafts with appropriate load factors applied for each limit state.

5. The weight of the drilled shafts is calculated from the proposed shaft top elevation presented on the assumed subsurface profile.

HWA GeoSciences Inc.	EDGEWATER CREEK BRIDGE REPLACEMENT EVERETT, WASHINGTON	WESTERN INTERIOR PIER (BH-3A) AXIAL SHAFT CAPACITIES 2-METER DIAMETER SHAFT

DRAWN BY: SKS
CHECKED BY: DJH
DATE: 05.07.20

FIGURE NO. **7C**

PROJECT NO. 2019-157-21

ASSUMED SUBSURFACE PROFILE

ELEVATION (feet)

Elevation	Layer
121'	FILL
111' / 108.5'	COLLUVIUM
	TRANSITIONAL BEDS
41'	
	WHIDBEY FORMATION
19.5'	

SERVICE LIMIT

AXIAL RESISTANCE (kips)

Legend:
- Nominal Side: 1-inch Settlement
- Nominal Base: 1-inch Settlement
- Factored Total: 1-inch Settlement

SERVICE LIMIT NOTES:

1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification is 1.0 for side resistance.

2. Settlements is based on a single shaft. No group action is considered.

3. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

STRENGTH LIMIT

AXIAL RESISTANCE (kips)

Legend:
- Nominal Side
- Nominal Base
- Factored Total

STRENGTH LIMIT NOTES:

1. Recommended resistance factors included in Factored Loads are 0.55 for cohesionless and 0.45 for cohesive for side resistance and 0.5 for cohesionless and 0.4 for cohesive for base resistance, as provided in AASHTO LRFD Bridge Design Specification.

2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.35 (per AASHTO LRFD Bridge Design Specification).

3. Recommended load factor of 1.25 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

EXTREME EVENT LIMIT

AXIAL RESISTANCE (kips)

Legend:
- Nominal Side
- Nominal Base
- Factored Total

EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification for both side and base resistance are 1.0 for compression and 0.8 for uplift.

2. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

GENERAL NOTES:

1. The analyses were performed based on guidelines included in the AASHTO LRFD Bridge Design Specification and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts.

2. Factored total shaft resistance shown on plots include the summation of the shaft's nominal side and base resistances multiplied by the appropriate resistance factors as noted above.

3. The nominal side and base resistance values presented do not include the resistance factors.

4. The nominal base and total factored axial capacities provided have been are reduced to account for the weight of drilled shafts with appropriate load factors applied for each limit state.

5. The weight of the drilled shafts is calculated from the proposed shaft top elevation presented on the assumed subsurface profile.

HWA GeoSciences Inc.	EDGEWATER CREEK BRIDGE REPLACEMENT EVERETT, WASHINGTON	WESTERN ABUTMENT (BH-4) AXIAL SHAFT CAPACITIES 2-METER DIAMETER SHAFT

DRAWN BY: SKS
CHECKED BY: DJH
DATE: 05.07.20

FIGURE NO. **7D**

PROJECT NO. 2019-157-21

ASSUMED SUBSURFACE PROFILE

ELEVATION (feet)

Elevation	Layer
125'	FILL
107.5'	TRANSITIONAL BEDS
45'	WHIDBEY FORMATION
24'	

SERVICE LIMIT

AXIAL RESISTANCE (kips)

ELEVATION (feet)

Legend:
- Nominal Side: 1-inch Settlement
- Nominal Base: 1-inch Settlement
- Factored Total: 1-inch Settlement

STRENGTH LIMIT

AXIAL RESISTANCE (kips)

ELEVATION (feet)

Legend:
- Nominal Side
- Nominal Base
- Factored Total

EXTREME EVENT LIMIT

AXIAL RESISTANCE (kips)

ELEVATION (feet)

Legend:
- Nominal Side
- Nominal Base
- Factored Total

SERVICE LIMIT NOTES:

1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification is 1.0 for side resistance.
2. Settlements is based on a single shaft. No group action is considered.
3. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

STRENGTH LIMIT NOTES:

1. Recommended resistance factors included in Factored Loads are 0.55 for cohesionless and 0.45 for cohesive for side resistance and 0.5 for cohesionless and 0.4 for cohesive for base resistance, as provided in AASHTO LRFD Bridge Design Specification.
2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.35 (per AASHTO LRFD Bridge Design Specification).
3. Recommended load factor of 1.25 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification for both side and base resistance are 1.0 for compression and 0.8 for uplift.
2. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

GENERAL NOTES:

1. The analyses were performed based on guidelines included in the AASHTO LRFD Bridge Design Specification and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts.

2. Factored total shaft resistance shown on plots include the summation of the shaft's nominal side and base resistances multiplied by the appropriate resistance factors as noted above.

3. The nominal side and base resistance values presented do not include the resistance factors.

4. The nominal base and total factored axial capacities provided have been are reduced to account for the weight of drilled shafts with appropriate load factors applied for each limit state.

5. The weight of the drilled shafts is calculated from the proposed shaft top elevation presented on the assumed subsurface profile.

ASSUMED SUBSURFACE PROFILE

ELEVATION (feet)

- 64' — FILL
- 62.5'
- COLLUVIUM
- 54'
- TRANSITIONAL BEDS
- 31'
- WHIDBEY FORMATION
- -12'

SERVICE LIMIT

AXIAL RESISTANCE (kips)

Legend:
- Nominal Side: 1-inch Settlement
- Nominal Base: 1-inch Settlement
- Factored Total: 1-inch Settlement

SERVICE LIMIT NOTES:

1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification is 1.0 for side resistance.
2. Settlements is based on a single shaft. No group action is considered.
3. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

STRENGTH LIMIT

AXIAL RESISTANCE (kips)

Legend:
- Nominal Side
- Nominal Base
- Factored Total

STRENGTH LIMIT NOTES:

1. Recommended resistance factors included in Factored Loads are 0.55 for cohesionless and 0.45 for cohesive for side resistance and 0.5 for cohesionless and 0.4 for cohesive for base resistance, as provided in AASHTO LRFD Bridge Design Specification.
2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.35 (per AASHTO LRFD Bridge Design Specification).
3. Recommended load factor of 1.25 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

EXTREME EVENT LIMIT

AXIAL RESISTANCE (kips)

Legend:
- Nominal Side
- Nominal Base
- Factored Total

EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification for both side and base resistance are 1.0 for compression and 0.8 for uplift.
2. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

GENERAL NOTES:

1. The analyses were performed based on guidelines included in the AASHTO LRFD Bridge Design Specification and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts.

2. Factored total shaft resistance shown on plots include the summation of the shaft's nominal side and base resistances multiplied by the appropriate resistance factors as noted above.

3. The nominal side and base resistance values presented do not include the resistance factors.

4. The nominal base and total factored axial capacities provided have been are reduced to account for the weight of drilled shafts with appropriate load factors applied for each limit state.

5. The weight of the drilled shafts is calculated from the proposed shaft top elevation presented on the assumed subsurface profile.

HWA GeoSciences Inc.

EDGEWATER CREEK BRIDGE REPLACEMENT EVERETT, WASHINGTON

EASTERN INTERIOR PIER (BH-2A) AXIAL SHAFT CAPACITIES 2.5-METER DIAMETER SHAFT

DRAWN BY	SKS
CHECKED BY	DJH
DATE	05.07.20

FIGURE NO. **8B**

PROJECT NO. 2019-157-21

ASSUMED SUBSURFACE PROFILE

ELEVATION (feet)

- 62'
- COLLUVIUM
- 50'
- TRANSITIONAL BEDS
- 37'
- WHIDBEY FORMATION
- -12.25'

SERVICE LIMIT

AXIAL RESISTANCE (kips)

Legend:
- Nominal Side: 1-inch Settlement
- Nominal Base: 1-inch Settlement
- Factored Total: 1-inch Settlement

ELEVATION (feet)

STRENGTH LIMIT

AXIAL RESISTANCE (kips)

Legend:
- Nominal Side
- Nominal Base
- Factored Total

ELEVATION (feet)

EXTREME EVENT LIMIT

AXIAL RESISTANCE (kips)

Legend:
- Nominal Side
- Nominal Base
- Factored Total

Add Downdrag Loads to Other Foundation Loads
(see Extreme Event Limit Note 2)

ELEVATION (feet)

SERVICE LIMIT NOTES:

1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification is 1.0 for side resistance.
2. Settlements is based on a single shaft. No group action is considered.
3. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

STRENGTH LIMIT NOTES:

1. Recommended resistance factors included in Factored Loads are 0.55 for cohesionless and 0.45 for cohesive for side resistance and 0.5 for cohesionless and 0.4 for cohesive for base resistance, as provided in AASHTO LRFD Bridge Design Specification.
2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.35 (per AASHTO LRFD Bridge Design Specification).
3. Recommended load factor of 1.25 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification for both side and base resistance are 1.0 for compression and 0.8 for uplift.
2. Unfactored static downdrag force, due to liquefaction-induced settlement, for each shaft is estimated to be **275 kips.** A load factor of 1.05 should be applied to all downdrag loads (Allen, 2005) to determine factored downdrag force.
3. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

GENERAL NOTES:

1. The analyses were performed based on guidelines included in the AASHTO LRFD Bridge Design Specification and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts.

2. Factored total shaft resistance shown on plots include the summation of the shaft's nominal side and base resistances multiplied by the appropriate resistance factors as noted above.

3. The nominal side and base resistance values presented do not include the resistance factors.

4. The nominal base and total factored axial capacities provided have been are reduced to account for the weight of drilled shafts with appropriate load factors applied for each limit state.

5. The weight of the drilled shafts is calculated from the proposed shaft top elevation presented on the assumed subsurface profile.

HWA GeoSciences Inc.

EDGEWATER CREEK BRIDGE REPLACEMENT EVERETT, WASHINGTON

WESTERN INTERIOR PIER (BH-3A) AXIAL SHAFT CAPACITIES 2.5-METER DIAMETER SHAFT

DRAWN BY: SKS
CHECKED BY: DJH
DATE: 05.07.20

FIGURE NO. **8C**

PROJECT NO. 2019-157-21

ASSUMED SUBSURFACE PROFILE

ELEVATION (feet)

- 121' FILL
- 111' / 108.5' COLLUVIUM
- TRANSITIONAL BEDS
- 41'
- WHIDBEY FORMATION
- 19.5'

SERVICE LIMIT

AXIAL RESISTANCE (kips)

ELEVATION (feet)

Legend:
- Nominal Side: 1-inch Settlement
- Nominal Base: 1-inch Settlement
- Factored Total: 1-inch Settlement

SERVICE LIMIT NOTES:

1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification is 1.0 for side resistance.

2. Settlements is based on a single shaft. No group action is considered.

3. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

STRENGTH LIMIT

AXIAL RESISTANCE (kips)

ELEVATION (feet)

Legend:
- Nominal Side
- Nominal Base
- Factored Total

STRENGTH LIMIT NOTES:

1. Recommended resistance factors included in Factored Loads are 0.55 for cohesionless and 0.45 for cohesive for side resistance and 0.5 for cohesionless and 0.4 for cohesive for base resistance, as provided in AASHTO LRFD Bridge Design Specification.

2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.35 (per AASHTO LRFD Bridge Design Specification).

3. Recommended load factor of 1.25 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

EXTREME EVENT LIMIT

AXIAL RESISTANCE (kips)

ELEVATION (feet)

Legend:
- Nominal Side
- Nominal Base
- Factored Total

EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification for both side and base resistance are 1.0 for compression and 0.8 for uplift.

2. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

GENERAL NOTES:

1. The analyses were performed based on guidelines included in the AASHTO LRFD Bridge Design Specification and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts.

2. Factored total shaft resistance shown on plots include the summation of the shaft's nominal side and base resistances multiplied by the appropriate resistance factors as noted above.

3. The nominal side and base resistance values presented do not include the resistance factors.

4. The nominal base and total factored axial capacities provided have been are reduced to account for the weight of drilled shafts with appropriate load factors applied for each limit state.

5. The weight of the drilled shafts is calculated from the proposed shaft top elevation presented on the assumed subsurface profile.

HWA GeoSciences Inc.

EDGEWATER CREEK
BRIDGE REPLACEMENT
EVERETT, WASHINGTON

WESTERN ABUTMENT (BH-4)
AXIAL SHAFT CAPACITIES
2.5-METER DIAMETER SHAFT

DRAWN BY SKS
CHECKED BY DJH
DATE 05.07.20

FIGURE NO. 8D

PROJECT NO. 2019-157-21

ASSUMED SUBSURFACE PROFILE

ELEVATION (feet)

- 125'
- FILL
- 107.5'
- TRANSITIONAL BEDS
- 45'
- WHIDBEY FORMATION
- 24'

SERVICE LIMIT

AXIAL RESISTANCE (kips)

ELEVATION (feet)

Legend:
- Nominal Side: 1-inch Settlement
- Nominal Base: 1-inch Settlement
- Factored Total: 1-inch Settlement

SERVICE LIMIT NOTES:

1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification is 1.0 for side resistance.

2. Settlements is based on a single shaft. No group action is considered.

3. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

STRENGTH LIMIT

AXIAL RESISTANCE (kips)

ELEVATION (feet)

Legend:
- Nominal Side
- Nominal Base
- Factored Total

STRENGTH LIMIT NOTES:

1. Recommended resistance factors included in Factored Loads are 0.55 for cohesionless and 0.45 for cohesive for side resistance and 0.5 for cohesionless and 0.4 for cohesive for base resistance, as provided in AASHTO LRFD Bridge Design Specification.

2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.35 (per AASHTO LRFD Bridge Design Specification).

3. Recommended load factor of 1.25 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

EXTREME EVENT LIMIT

AXIAL RESISTANCE (kips)

ELEVATION (feet)

Legend:
- Nominal Side
- Nominal Base
- Factored Total

EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification for both side and base resistance are 1.0 for compression and 0.8 for uplift.

2. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

GENERAL NOTES:

1. The analyses were performed based on guidelines included in the AASHTO LRFD Bridge Design Specification and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts.

2. Factored total shaft resistance shown on plots include the summation of the shaft's nominal side and base resistances multiplied by the appropriate resistance factors as noted above.

3. The nominal side and base resistance values presented do not include the resistance factors.

4. The nominal base and total factored axial capacities provided have been are reduced to account for the weight of drilled shafts with appropriate load factors applied for each limit state.

5. The weight of the drilled shafts is calculated from the proposed shaft top elevation presented on the assumed subsurface profile.

HWA GeoSciences Inc.

| EDGEWATER CREEK BRIDGE REPLACEMENT EVERETT, WASHINGTON | EASTERN ABUTMENT (BH-1) AXIAL SHAFT CAPACITIES 3-METER DIAMETER SHAFT | DRAWN BY: SKS
CHECKED BY: DJH
DATE: 05.07.20 | FIGURE NO. 9A
PROJECT NO. 2019-157-21 |

ASSUMED SUBSURFACE PROFILE

SERVICE LIMIT

AXIAL RESISTANCE (kips)

Legend:
- Nominal Side: 1-inch Settlement
- Nominal Base: 1-inch Settlement
- Factored Total: 1-inch Settlement

SERVICE LIMIT NOTES:

1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification is 1.0 for side resistance.

2. Settlements is based on a single shaft. No group action is considered.

3. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

STRENGTH LIMIT

AXIAL RESISTANCE (kips)

Legend:
- Nominal Side
- Nominal Base
- Factored Total

STRENGTH LIMIT NOTES:

1. Recommended resistance factors included in Factored Loads are 0.55 for cohesionless and 0.45 for cohesive for side resistance and 0.5 for cohesionless and 0.4 for cohesive for base resistance, as provided in AASHTO LRFD Bridge Design Specification.

2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.35 (per AASHTO LRFD Bridge Design Specification).

3. Recommended load factor of 1.25 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

EXTREME EVENT LIMIT

AXIAL RESISTANCE (kips)

Legend:
- Nominal Side
- Nominal Base
- Factored Total

EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification for both side and base resistance are 1.0 for compression and 0.8 for uplift.

2. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

GENERAL NOTES:

1. The analyses were performed based on guidelines included in the AASHTO LRFD Bridge Design Specification and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts.

2. Factored total shaft resistance shown on plots include the summation of the shaft's nominal side and base resistances multiplied by the appropriate resistance factors as noted above.

3. The nominal side and base resistance values presented do not include the resistance factors.

4. The nominal base and total factored axial capacities provided have been are reduced to account for the weight of drilled shafts with appropriate load factors applied for each limit state.

5. The weight of the drilled shafts is calculated from the proposed shaft top elevation presented on the assumed subsurface profile.

HWA HWA GeoSciences Inc.	EDGEWATER CREEK BRIDGE REPLACEMENT EVERETT, WASHINGTON	EASTERN INTERIOR PIER (BH-2A) AXIAL SHAFT CAPACITIES 3-METER DIAMETER SHAFT

DRAWN BY SKS	FIGURE NO. **9B**
CHECKED BY DJH	
DATE 05.07.20	PROJECT NO. 2019-157-21

ASSUMED SUBSURFACE PROFILE

ELEVATION (feet)

Elevation	Unit
62'	COLLUVIUM
50'	TRANSITIONAL BEDS
37'	WHIDBEY FORMATION
-12.25'	

SERVICE LIMIT

AXIAL RESISTANCE (kips)

Legend:
- Nominal Side: 1-inch Settlement
- Nominal Base: 1-inch Settlement
- Factored Total: 1-inch Settlement

SERVICE LIMIT NOTES:

1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification is 1.0 for side resistance.

2. Settlements is based on a single shaft. No group action is considered.

3. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

STRENGTH LIMIT

AXIAL RESISTANCE (kips)

Legend:
- Nominal Side
- Nominal Base
- Factored Total

STRENGTH LIMIT NOTES:

1. Recommended resistance factors included in Factored Loads are 0.55 for cohesionless and 0.45 for cohesive for side resistance and 0.5 for cohesionless and 0.4 for cohesive for base resistance, as provided in AASHTO LRFD Bridge Design Specification.

2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.35 (per AASHTO LRFD Bridge Design Specification).

3. Recommended load factor of 1.25 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

EXTREME EVENT LIMIT

AXIAL RESISTANCE (kips)

Add Downdrag Loads to Other Foundation Loads (see Extreme Event Limit Note 2)

Legend:
- Nominal Side
- Nominal Base
- Factored Total

EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification for both side and base resistance are 1.0 for compression and 0.8 for uplift.

2. Unfactored static downdrag force, due to liquefaction-induced settlement, for each shaft is estimated to be **325 kips.** A load factor of 1.05 should be applied to all downdrag loads (Allen, 2005) to determine factored downdrag force.

3. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

GENERAL NOTES:

1. The analyses were performed based on guidelines included in the AASHTO LRFD Bridge Design Specification and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts.

2. Factored total shaft resistance shown on plots include the summation of the shaft's nominal side and base resistances multiplied by the appropriate resistance factors as noted above.

3. The nominal side and base resistance values presented do not include the resistance factors.

4. The nominal base and total factored axial capacities provided have been are reduced to account for the weight of drilled shafts with appropriate load factors applied for each limit state.

5. The weight of the drilled shafts is calculated from the proposed shaft top elevation presented on the assumed subsurface profile.

HWA GeoSciences Inc.

EDGEWATER CREEK BRIDGE REPLACEMENT EVERETT, WASHINGTON

WESTERN INTERIOR PIER (BH-3A) AXIAL SHAFT CAPACITIES 3-METER DIAMETER SHAFT

DRAWN BY	SKS
CHECKED BY	DJH
DATE	05.07.20

FIGURE NO. **9C**

PROJECT NO. 2019-157-21

ASSUMED SUBSURFACE PROFILE

ELEVATION (feet)

121' — FILL
111' — COLLUVIUM
108.5'
TRANSITIONAL BEDS
41'
WHIDBEY FORMATION
19.5'

SERVICE LIMIT

AXIAL RESISTANCE (kips)

Legend:
- Nominal Side: 1-inch Settlement
- Nominal Base: 1-inch Settlement
- Factored Total: 1-inch Settlement

STRENGTH LIMIT

AXIAL RESISTANCE (kips)

Legend:
- Nominal Side
- Nominal Base
- Factored Total

EXTREME EVENT LIMIT

AXIAL RESISTANCE (kips)

Legend:
- Nominal Side
- Nominal Base
- Factored Total

SERVICE LIMIT NOTES:

1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification is 1.0 for side resistance.

2. Settlements is based on a single shaft. No group action is considered.

3. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

STRENGTH LIMIT NOTES:

1. Recommended resistance factors included in Factored Loads are 0.55 for cohesionless and 0.45 for cohesive for side resistance and 0.5 for cohesionless and 0.4 for cohesive for base resistance, as provided in AASHTO LRFD Bridge Design Specification.

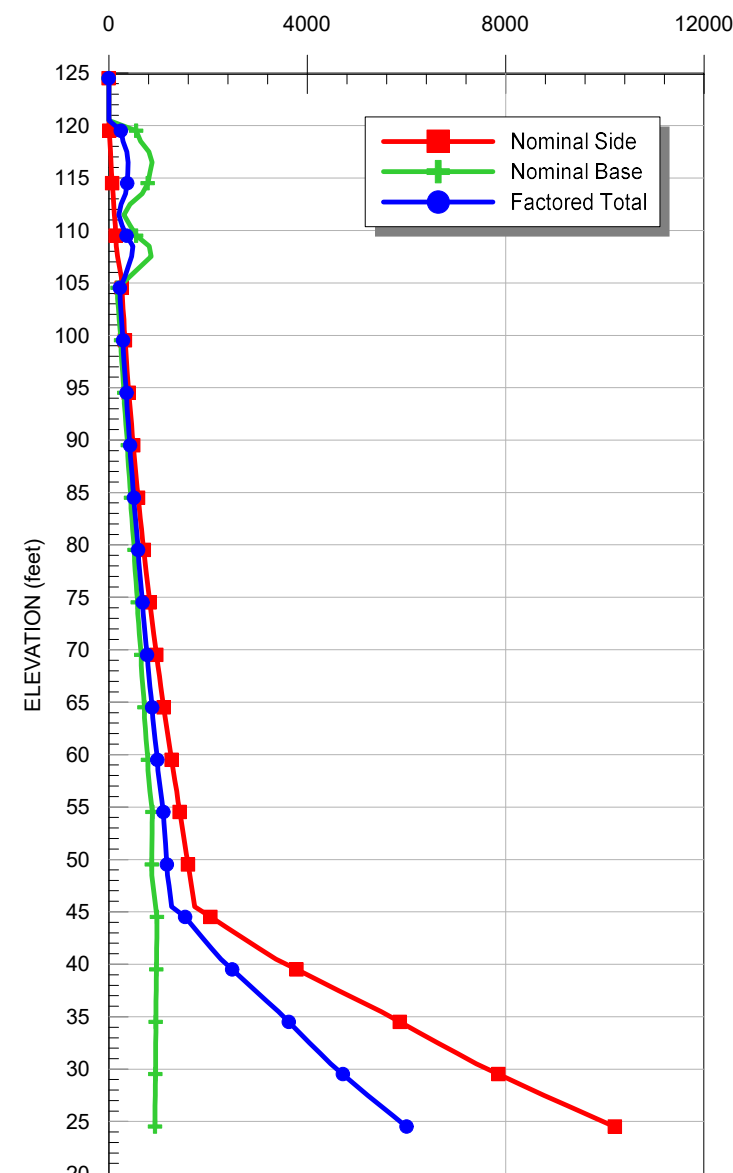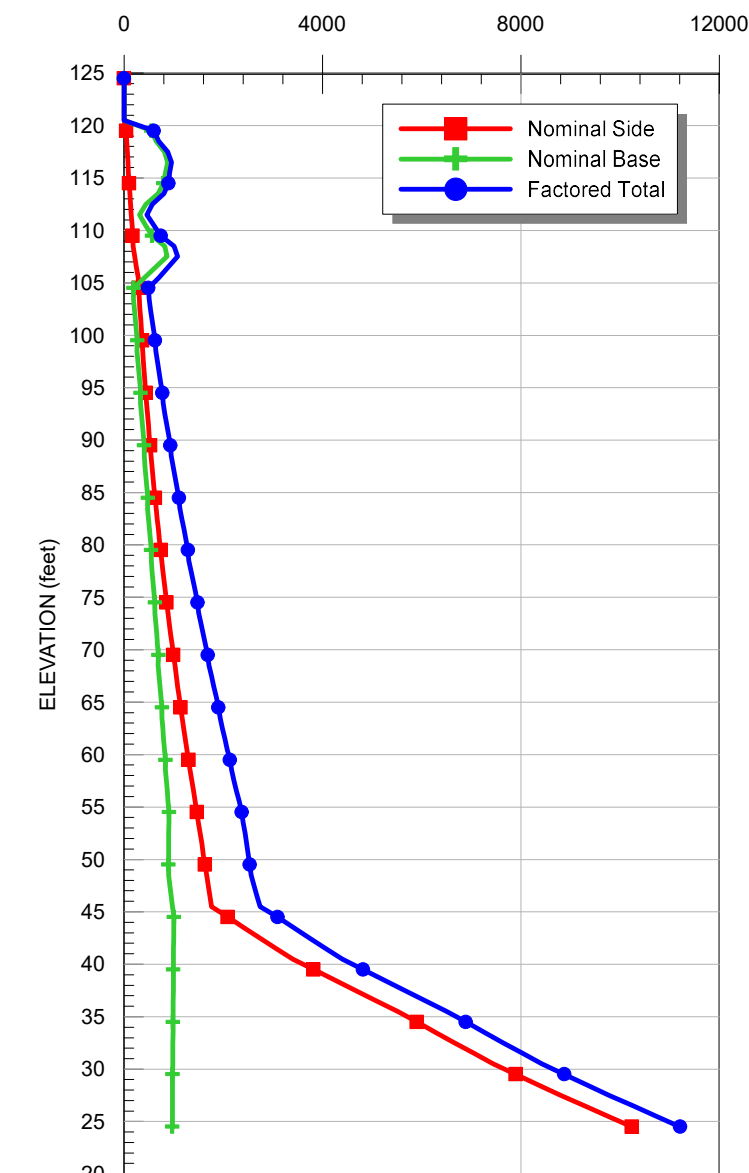2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.35 (per AASHTO LRFD Bridge Design Specification).

3. Recommended load factor of 1.25 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification for both side and base resistance are 1.0 for compression and 0.8 for uplift.

2. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

GENERAL NOTES:

1. The analyses were performed based on guidelines included in the AASHTO LRFD Bridge Design Specification and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts.

2. Factored total shaft resistance shown on plots include the summation of the shaft's nominal side and base resistances multiplied by the appropriate resistance factors as noted above.

3. The nominal side and base resistance values presented do not include the resistance factors.

4. The nominal base and total factored axial capacities provided have been are reduced to account for the weight of drilled shafts with appropriate load factors applied for each limit state.

5. The weight of the drilled shafts is calculated from the proposed shaft top elevation presented on the assumed subsurface profile.

HWA GeoSciences Inc.	EDGEWATER CREEK BRIDGE REPLACEMENT EVERETT, WASHINGTON	WESTERN ABUTMENT (BH-4) AXIAL SHAFT CAPACITIES 3-METER DIAMETER SHAFT

DRAWN BY: SKS
CHECKED BY: DJH
DATE: 05.07.20

FIGURE NO. 9D
PROJECT NO. 2019-157-21

SOILS LEGEND

- ▢ COLLUVIUM
- ▢ SATURATED COLLUVIUM
- ▢ LIQUEFIED COLLUVIUM
- ▢ TRANSITIONAL BEDS
- ▢ WHIDBEY FORMATION

GENERAL NOTES:

1. All pressures shown are in units of pounds per square foot (psf).

2. Shaft heights are in units of feet.

3. Passive pressures applied to shaft cap should be multiplied by 1.5 times the width of the shaft cap.

4. Passive pressures applied to drilled shafts should be multiplied by 3 times the diameter of the shafts.

5. Pressures assume a maximum extension of shaft caps limited to 1 feet beyond outer edge of drilled shaft.

6. Assume shaft cap thickness of 4 feet.

PIER NEAR BORING BH-03/03A

EXISTING GROUND SURFACE

SHAFT CAP

EL. 71.4 FEET

35° SLOPE

35° SLOPE

500 psf

2,150 psf

ELEVATION IN FEET

COLLUVIUM

17'

4'

18'

TRANSITIONAL BEDS

COARSE-GRAINED GLACIALLY CONSOLIDATED WHIDBEY FORMATION
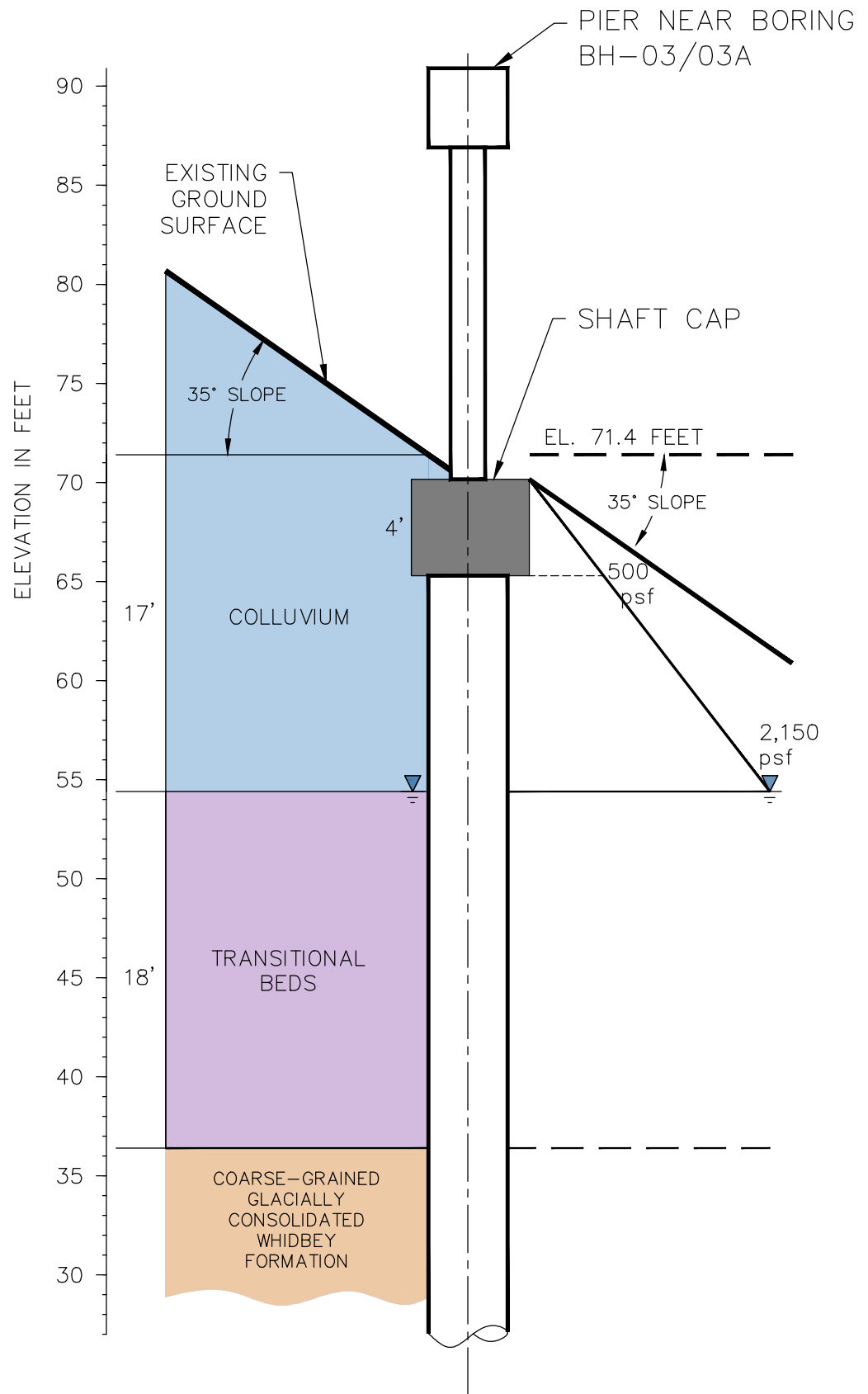
NOT TO SCALE

LIQUEFIABLE PRESSURE DIAGRAM

EDGEWATER BRIDGE REPLACEMENT
EVERETT, WASHINGTON

FIGURE NO.:
10

DRAWN BY: CF   CHECK BY: SKS
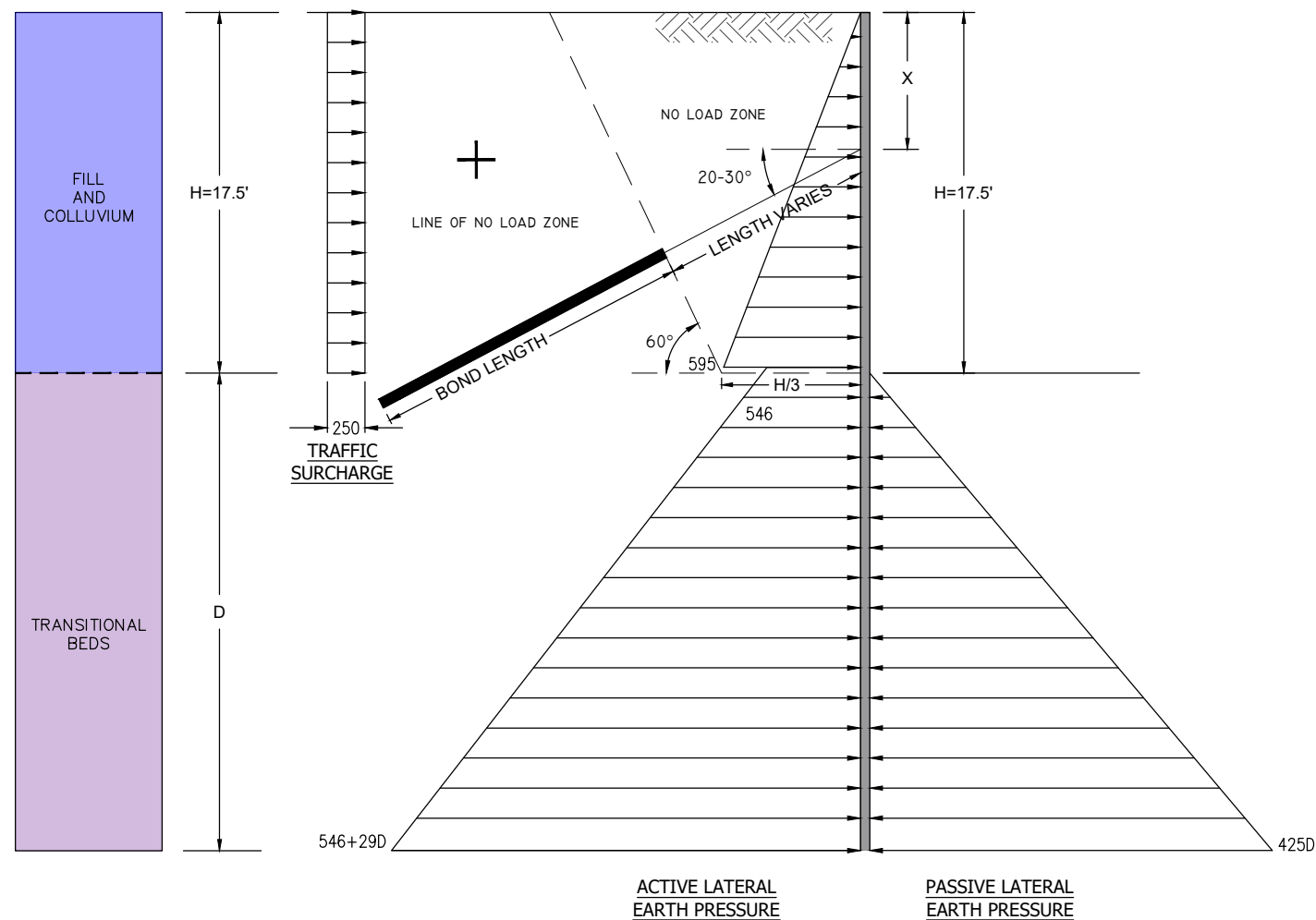
PROJECT #
2019-157-21

HWA GeoSciences Inc.
DBE/MWBE

STRENGTH & SERVICE LIMIT STATE (STATIC)

Diagram labels (left): FILL AND COLLUVIUM, TRANSITIONAL BEDS, H=17.5', D, NO LOAD ZONE, LINE OF NO LOAD ZONE, 20-30°, X, H=17.5', LENGTH VARIES, BOND LENGTH, 60°, 595, 250, 546, H/3, 546+29D, 425D, TRAFFIC SURCHARGE, ACTIVE LATERAL EARTH PRESSURE, PASSIVE LATERAL EARTH PRESSURE

EXTREME (EQ) LIMIT STATE (SEISMIC)

Diagram labels (right): NO LOAD ZONE, LINE OF NO LOAD ZONE, 20-30°, X, H=17.5', LENGTH VARIES, BOND LENGTH, 60°, 595, 168, 250, 546, H/3, 546+29D, 370D, TRAFFIC SURCHARGE (SEISMIC), ACTIVE (SEISMIC), ACTIVE LATERAL EARTH PRESSURE, PASSIVE-SEISMIC LATERAL EARTH PRESSURE

General

1. All the pressures shown are in the units of pounds per square foot (psf).
2. Lateral earth pressures provided herein are based on active earth pressures and should be used for the design of the retaining walls where the wall is free to displace laterally at least 0.001H, where H is the retained height of the wall.
3. All the earth pressures provided are ultimate (unfactored), the appropriate load and resistance factors should be applied for each load state.
4. All earth pressures assume a flat back slope.
5. Passive pressures shown assume that the soils above the toe of the slope do not contribute to passive resistance.
6. All active earth pressures acting on the retained portion of the wall (above the base of the wall) should be applied across the pile spacing.
7. All active earth pressures acting below the retained portion of the wall (below the base of the wall) should be applied over one pile shaft diameter.
8. Surcharge load should be equal to factored Dead and Live Load including equipment, traffic, etc.
9. Soldier pile elements should extend to a minimum depth of 50 feet below proposed roadway surface in order to provide the desired slope stabilization.

Strength and Service State Design

1. For strength limit state design, a resistance factor (φ) of 0.75 should be applied to the passive earth pressures shown.
2. For service limit state design, a resistance factor (φ) of 1.0 should be applied to the passive earth pressures shown.
3. All passive earth pressures should be applied over two shaft diameters.

Extreme Limit State Design

1. All passive earth pressures should be applied over two shaft diameters.
2. Lateral earth pressures presented under Extreme Limit State include active plus seismic on the retained side and passive plus seismic on the cut side of the wall.

* Traffic surcharge pressure under seismic conditions is based on commentary in AASHTO 8th Edition, Section 3.4.1.

Soldier Pile Wall with One Row of Tiebacks

1. The tiebacks should be angled downward 20 to 30 degrees from the horizontal.
2. Although the full length of the tieback is grouted, a bond breaker such as a grease coating protected by plastic sheathing should be used in the no-load zone.
3. The ultimate capacity of the tieback should be estimated based on 1,000 psf of anchor surface area beyond the no-load zone.
4. Actual tieback design, including grout mix design, anchor length, tendon design, and drilling and grouting methods should be designed by the contractor. The contractor should then be responsible for achieving the design capacity of each anchor.

NOT TO SCALE

HWA GeoSciences Inc. DBE/MWBE

EDGEWATER BRIDGE REPLACEMENT
EVERETT, WASHINGTON

LATERAL EARTH PRESSURES FOR SOLDIER PILE WALL

DRAWN BY: CF
CHECK BY: SKS

FIGURE NO.: 11
PROJECT NO.: 2019-157-21

TYPICAL SECTION
CONSTRUCTION
SURCHARGE

Q

FILL
AND
COLLUVIUM

7.5'

10'

TRANSITIONAL
BEDS

D

H=10'

255
235

0.28Q

330+29D

425(D−2)

ACTIVE LATERAL
EARTH PRESSURE

PASSIVE LATERAL
EARTH PRESSURE

STRENGTH & SERVICE LIMIT STATE (STATIC)

H=10'

72
70

91

SEISMIC LATERAL
EARTH PRESSURE

255
235

330+29D

370(D−2)

ACTIVE LATERAL
EARTH PRESSURE

PASSIVE-SEISMIC
LATERAL EARTH
PRESSURE
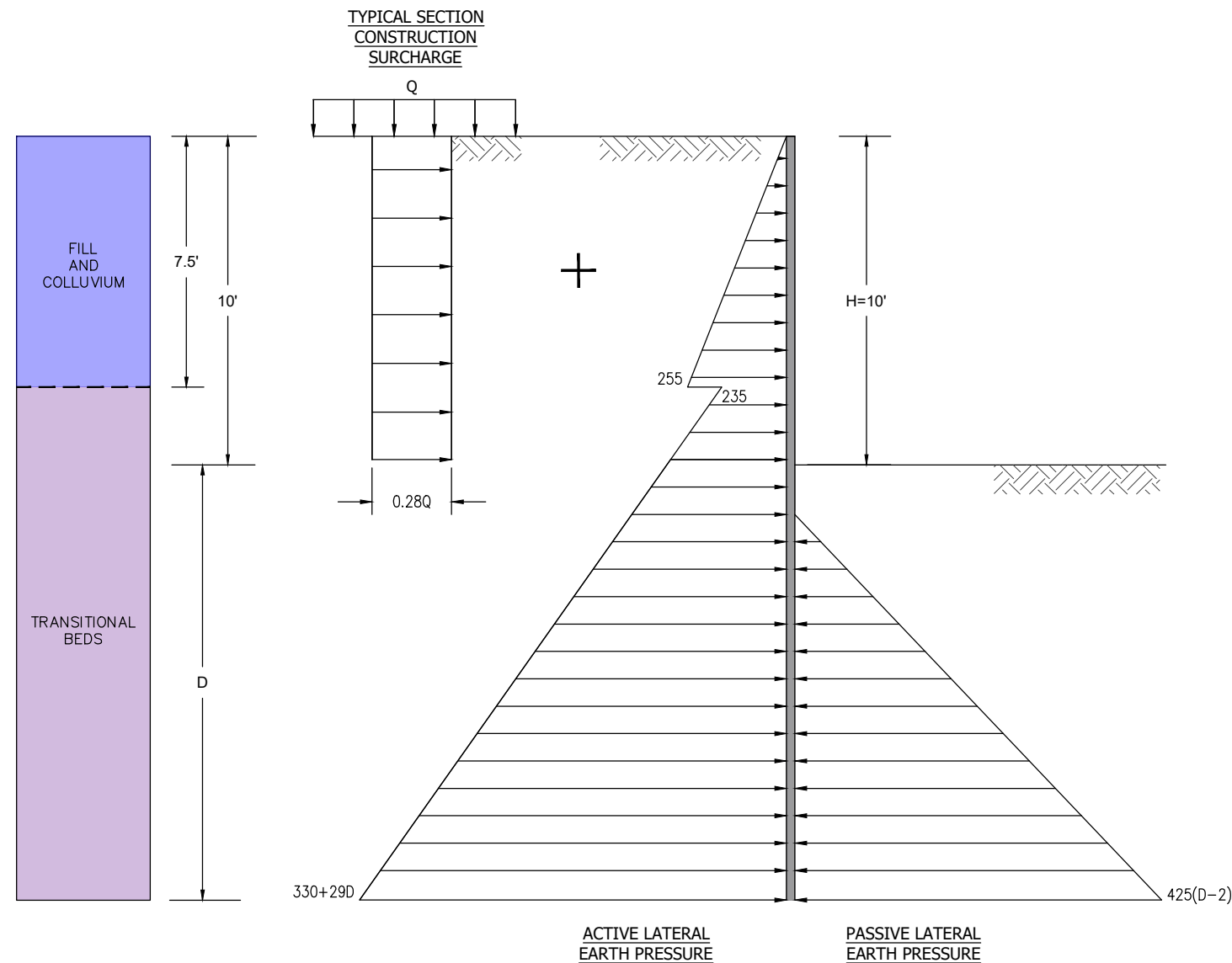
EXTREME (EQ) LIMIT STATE (SEISMIC)

General
1. All the pressures shown are in the units of pounds per square foot (psf).
2. Lateral earth pressures provided herein are based on active earth pressures and should be used for the design of the retaining walls where the wall is free to displace laterally at least 0.001H, where H is the retained height of the wall.
3. All the earth pressures provided are ultimate (unfactored), the appropriate load and resistance factors should be applied for each load state.
4. All earth pressures assume a flat back slope.
5. All active earth pressures acting on the retained portion of the wall (above the base of the wall) should be applied across the pile spacing.
6. All active earth pressures acting below the retained portion of the wall (below the base of the wall) should be applied over one pile shaft diameter.
7. Earth pressure diagrams assume no surcharge applied to the top of the wall after completion of construction.

Strength and Service State Design
1. For strength limit state design, a resistance factor (ϕ) of 0.75 should be applied to the passive earth pressures shown.
2. For service limit state design, a resistance factor (ϕ) of 1.0 should be applied to the passive earth pressures shown.
3. All passive earth pressures should be applied over two shaft diameters.

Extreme Limit State Design
1. All passive earth pressures should be applied over two shaft diameters.
2. Lateral earth pressures presented under Extreme Limit State include active plus seismic on the retained side and passive plus seismic on the cut side of the wall.

* Traffic surcharge pressure under seismic conditions is based on commentary in AASHTO 8th Edition, Section 3.4.1.

NOT TO SCALE

APPENDIX A

FIELD EXPLORATION

RELATIVE DENSITY OR CONSISTENCY VERSUS SPT N-VALUE

COHESIONLESS SOILS			COHESIVE SOILS		
Density	N (blows/ft)	Approximate Relative Density(%)	Consistency	N (blows/ft)	Approximate Undrained Shear Strength (psf)
Very Loose	0 to 4	0 - 15	Very Soft	0 to 2	<250
Loose	4 to 10	15 - 35	Soft	2 to 4	250 - 500
Medium Dense	10 to 30	35 - 65	Medium Stiff	4 to 8	500 - 1000
Dense	30 to 50	65 - 85	Stiff	8 to 15	1000 - 2000
Very Dense	over 50	85 - 100	Very Stiff	15 to 30	2000 - 4000
			Hard	over 30	>4000

USCS SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS					GROUP DESCRIPTIONS
Coarse Grained Soils	Gravel and Gravelly Soils	Clean Gravel (little or no fines)		GW	Well-graded GRAVEL
				GP	Poorly-graded GRAVEL
	More than 50% of Coarse Fraction Retained on No. 4 Sieve	Gravel with Fines (appreciable amount of fines)		GM	Silty GRAVEL
				GC	Clayey GRAVEL
More than 50% Retained on No. 200 Sieve Size	Sand and Sandy Soils	Clean Sand (little or no fines)		SW	Well-graded SAND
				SP	Poorly-graded SAND
	50% or More of Coarse Fraction Passing No. 4 Sieve	Sand with Fines (appreciable amount of fines)		SM	Silty SAND
				SC	Clayey SAND
Fine Grained Soils	Silt and Clay	Liquid Limit Less than 50%		ML	SILT
				CL	Lean CLAY
				OL	Organic SILT/Organic CLAY
50% or More Passing No. 200 Sieve Size	Silt and Clay	Liquid Limit 50% or More		MH	Elastic SILT
				CH	Fat CLAY
				OH	Organic SILT/Organic CLAY
Highly Organic Soils				PT	PEAT

TEST SYMBOLS

%F	Percent Fines
AL	Atterberg Limits: PL = Plastic Limit / LL = Liquid Limit
CBR	California Bearing Ratio
CN	Consolidation
DD	Dry Density (pcf)
DS	Direct Shear
GS	Grain Size Distribution
K	Permeability
MD	Moisture/Density Relationship (Proctor)
MR	Resilient Modulus
PID	Photoionization Device Reading
PP	Pocket Penetrometer — Approx. Compressive Strength (tsf)
SG	Specific Gravity
TC	Triaxial Compression
TV	Torvane — Approx. Shear Strength (tsf)
UC	Unconfined Compression

SAMPLE TYPE SYMBOLS

- 2.0" OD Split Spoon (SPT) (140 lb. hammer with 30 in. drop)
- Shelby Tube
- 3-1/4" OD Split Spoon with Brass Rings
- Small Bag Sample
- Large Bag (Bulk) Sample
- Core Run
- Non-standard Penetration Test (3.0" OD split spoon)

GROUNDWATER SYMBOLS

- ▽ Groundwater Level (measured at time of drilling)
- ▼ Groundwater Level (measured in well or open hole after water level stabilized)

COMPONENT DEFINITIONS

COMPONENT	SIZE RANGE
Boulders	Larger than 12 in
Cobbles	3 in to 12 in
Gravel	3 in to No 4 (4.5mm)
Coarse gravel	3 in to 3/4 in
Fine gravel	3/4 in to No 4 (4.5mm)
Sand	No. 4 (4.5 mm) to No. 200 (0.074 mm)
Coarse sand	No. 4 (4.5 mm) to No. 10 (2.0 mm)
Medium sand	No. 10 (2.0 mm) to No. 40 (0.42 mm)
Fine sand	No. 40 (0.42 mm) to No. 200 (0.074 mm)
Silt and Clay	Smaller than No. 200 (0.074mm)

COMPONENT PROPORTIONS

PROPORTION RANGE	DESCRIPTIVE TERMS
< 5%	Clean
5 - 12%	Slightly (Clayey, Silty, Sandy)
12 - 30%	Clayey, Silty, Sandy, Gravelly
30 - 50%	Very (Clayey, Silty, Sandy, Gravelly)
Components are arranged in order of increasing quantities.	

NOTES: Soil classifications presented on exploration logs are based on visual and laboratory observation.
Soil descriptions are presented in the following general order:

Density/consistency, color, modifier (if any) GROUP NAME, additions to group name (if any), moisture content. Proportion, gradation, and angularity of constituents, additional comments. (GEOLOGIC INTERPRETATION)

Please refer to the discussion in the report text as well as the exploration logs for a more complete description of subsurface conditions.

MOISTURE CONTENT

DRY	Absence of moisture, dusty, dry to the touch.
MOIST	Damp but no visible water.
WET	Visible free water, usually soil is below water table.

HWA GeoSciences Inc.

Edgewater Creek Bridge
Everett, Washington

LEGEND OF TERMS AND SYMBOLS USED ON EXPLORATION LOGS

PROJECT NO.: 2019-157-21   FIGURE: A-1

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Mud Rotary, Dietrich D-120 Truck Rig
SAMPLING METHOD: SPT w/ Autohammer
LOCATION: See Figure 2

DATE STARTED: 3/20/2020
DATE COMPLETED: 3/20/2020
LOGGED BY: A.Mahmoud
SURFACE ELEVATION: 125.0 ± feet

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE / SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
0		GM	Asphalt concrete pavement.			
			Medium dense, brown, silty, sandy, GRAVEL, moist. **(FILL)**			
				S-1	10-13-12	
5		SM	Medium dense, olive brown, slightly gravelly, slightly silty, SAND, moist.	S-2	9-9-7	
			Becomes rust-mottled olive brown.	S-3a	16-17-15	GS
		GP / GM	Medium dense, olive brown, very sandy, slightly silty, GRAVEL, moist.	S-3b		GS
10			No recovery.	S-4	19-20-21	
		SP / SC	Medium dense, rust-mottled gray, slightly gravelly, clayey, SAND, moist.	S-5	10-11-12	
15		ML	Medium dense, gray, sandy, SILT, moist. Trace gravel.	S-6	5-18-12	
		GP	Very dense, gray, sandy, GRAVEL, wet. Trace interbedded clay lenses. **(TRANSITIONAL BEDS)**	S-7	19-50/6"	
20		CL	Hard, gray, sandy, CLAY, moist. Trace gravel.	S-8	33-50/6"	
25			Becomes dark gray. Blocky nature and laminated.	S-9	16-18-24	
30						

Standard Penetration Test
(140 lb. weight, 30" drop)
▲ Blows per foot

ELEVATION2 (feet)

Water Content (%)
Plastic Limit ⊢—●—⊣ Liquid Limit
Natural Water Content

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

Edgewater Creek Bridge
Everett, Washington

BORING:
BH-1

PAGE: 1 of 4

PROJECT NO.: 2019-157-21   FIGURE: A-2

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Mud Rotary, Dietrich D-120 Truck Rig
SAMPLING METHOD: SPT w/ Autohammer
LOCATION: See Figure 2

DATE STARTED: 3/20/2020
DATE COMPLETED: 3/20/2020
LOGGED BY: A.Mahmoud
SURFACE ELEVATION: 125.0 ± feet

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
30		ML	Hard, gray, SILT, moist.		S-10	11-18-24	AL %F
35		CL	Hard, rust-mottled gray, CLAY, moist.		S-11	18-26-30	
40		ML	Hard, rust-mottled olive brown, sandy, SILT, moist. Interbedded with clay.		S-12	18-28-40	GS
45			Becomes grayish brown.		S-13	30-50/6"	
50		CL	Hard, gray, CLAY, moist. Laminated and interbedded with sand.		S-14	28-50/6"	
55		ML	Very dense, dark gray, sandy, SILT, moist.		S-15	25-30-41	GS
60							

Standard Penetration Test
(140 lb. weight, 30" drop)
▲ Blows per foot

GROUNDWATER

ELEVATION (feet): 90, 85, 80, 75, 70

Water Content (%)

Plastic Limit ——●—— Liquid Limit
Natural Water Content

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GeoSciences Inc.

Edgewater Creek Bridge
Everett, Washington

BORING:
BH-1

PAGE: 2 of 4

PROJECT NO.: 2019-157-21   FIGURE: A-3

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Mud Rotary, Dietrich D-120 Truck Rig
SAMPLING METHOD: SPT w/ Autohammer
LOCATION: See Figure 2

DATE STARTED: 3/20/2020
DATE COMPLETED: 3/20/2020
LOGGED BY: A.Mahmoud
SURFACE ELEVATION: 125.0 ± feet

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
60		CL	Hard, gray, CLAY, moist. Laminated and interbedded with sand and non-plastic silt.	⊠	S-16	26-27-39	
65				⊠	S-17	25-50/6"	AL %F
70				⊠	S-18	28-50/6"	
75				⊠	S-19	35-50/6"	
80		SM	Very dense, dark gray, silty, SAND, moist. Interbedded with clay. **(WHIDBEY FORMATION)**	⊠	S-20	18-25-42	
85				⊠	S-21	28-50/6"	
90							

Standard Penetration Test
(140 lb. weight, 30" drop)
▲ Blows per foot

Plastic Limit ├──●──┤ Liquid Limit
Natural Water Content

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

Edgewater Creek Bridge
Everett, Washington

BORING:
BH-1

PAGE: 3 of 4

PROJECT NO.: 2019-157-21   FIGURE: A-4

BORING-DSM 2019-157-21.GPJ 6/24/20

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Mud Rotary, Dietrich D-120 Truck Rig
SAMPLING METHOD: SPT w/ Autohammer
LOCATION: See Figure 2

DATE STARTED: 3/20/2020
DATE COMPLETED: 3/20/2020
LOGGED BY: A.Mahmoud
SURFACE ELEVATION: 125.0 ± feet

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
90					S-22	30-32-43	
95			Becomes very dark gray. Trace fine gravel and broken fragments of macoma (clam) sea shells.		S-23	31-50/5"	GS
100					S-24	44-50/6"	

Borehole terminated at 101 feet below ground surface (bgs).
Borehole abandoned with 3/8-inch bentonite chips.

Standard Penetration Test
(140 lb. weight, 30" drop)
▲ Blows per foot

Groundwater / Elevation (feet) axis: 0 10 20 30 40 50

Water Content (%) axis: 0 20 40 60 80 100

Plastic Limit ├──●──┤ Liquid Limit
Natural Water Content

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

Edgewater Creek Bridge
Everett, Washington

BORING:
BH-1

PAGE: 4 of 4

PROJECT NO.: 2019-157-21   FIGURE: A-5

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Mud Rotary, Dietrich D-120 Truck Rig
SAMPLING METHOD: SPT w/ Autohammer
LOCATION: See Figure 2

DATE STARTED: 3/19/2020
DATE COMPLETED: 3/19/2020
LOGGED BY: A.Mahmoud
SURFACE ELEVATION: 64.0 ± feet

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE / SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
0			Lowered casing through corehole in concrete deck to ground surface, about 55 feet below. Casing sunk to approx. 10 feet, before start of drilling. No sample collected. **(COLLUVIUM)**				
10		CH	Very stiff, olive brown, CLAY, wet. Rust banded, and interbedded with fine sand. **(TRANSITIONAL BEDS)**	S-1	7-11-10		
15				S-2	6-7-11		
20			Becomes hard, yellow brown. Rust banded and laminated, blocky nature.	S-3	15-18-27		
25			Becomes rust-mottled gray. Interbedded with fine sand, 2 inch layers of silt.	S-4	10-16-32		
30							

Standard Penetration Test
(140 lb. weight, 30" drop)
▲ Blows per foot

ELEVATION (feet): 60, 55, 50, 45, 40, 35

Water Content (%): 0, 20, 40, 60, 80, 100

Plastic Limit ├──●──┤ Liquid Limit
Natural Water Content

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GeoSciences Inc.

Edgewater Creek Bridge
Everett, Washington

BORING:
BH-2
PAGE: 1 of 2

PROJECT NO.: 2019-157-21   FIGURE: A-6

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Mud Rotary, Dietrich D-120 Truck Rig
SAMPLING METHOD: SPT w/ Autohammer
LOCATION: See Figure 2

DATE STARTED: 3/19/2020
DATE COMPLETED: 3/19/2020
LOGGED BY: A.Mahmoud
SURFACE ELEVATION: 64.0 ± feet

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)
30		ML	Dense, gray, fine to medium sand, SILT, moist. Interbedded clay lenses, blocky nature.	⊠	S-5	19-16-22
35		SC	Dense, gray, clean SAND, moist. Interbedded clay lenses, trace shell fragments.	⊠	S-6a	11-14-25
		CL	**(WHIDBEY FORMATION)**	⊠	S-6b	
			Hard, gray, CLAY, moist. Interbedded with fine sand.			

Borehole terminated at 36.5 feet due to complete loss of drilling fluid from a depth of approx. 33 feet.
Borehole abandoned with bentonite chips.

Standard Penetration Test
(140 lb. weight, 30" drop)
▲ Blows per foot

Water Content (%)

Plastic Limit ├──●──┤ Liquid Limit
Natural Water Content

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GeoSciences Inc.

Edgewater Creek Bridge
Everett, Washington

BORING:
BH-2
PAGE: 2 of 2

PROJECT NO.: 2019-157-21   FIGURE: A-7

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Sonic (4.5 inch), GeoProbe 8140 LC
SAMPLING METHOD: Sonic, SPT w/Autohammer, and D&M w/Autohammer
LOCATION: See Figure 2

DATE STARTED: 3/31/2020
DATE COMPLETED: 4/1/2020
LOGGED BY: B. Thurber
SURFACE ELEVATION: 64.0 ± feet

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
0		ML	Lowered casing through corehole in concrete deck to ground surface, about 60 feet below.		S-1	0-0-0	
			Very soft, gray, SILT, moist. Scattered wood chips, abundant organics.				
			(TOPSOIL)				
			Very soft, olive gray, SILT, moist.				
			(COLLUVIUM)				
5			6-inch casing sunk to nearly 10 feet below ground surface. No samples collected from 1.5 to 10 feet.				
10		CH	Hard, rust-banded grayish-brown to olive-brown, SILT and CLAY, interbedded with fine sand, wet. Finely bedded to laminated.		S-2a S-2b S-2c	10-13-20	
			(TRANSITIONAL BEDS)				
			All remaining sample runs disturbed by sonic drilling. Scattered fine gravel, approx. 1/4 inch grain size. Rust banding along sand partings.		S-3		
15			Becomes very stiff.		S-4	7-12-13	AL %F
					S-5		
20		ML	Very stiff, rust-banded gray, SILT, wet. Observed 2 dark greenish-gray fine sand beds, each approx. 1/4 inch thick. Fine, blocky texture.		S-6	11-11-13	
			Finely interbedded with clay and dark green fine sand.		S-7		
			Laminations to approx. 3/16 inch. One 3/8 inch gray sand bed observed.				
25		ML	Hard and very dense, gray, SILT, CLAY, and fine SAND, wet. Sand is dark green to approx. 25.3 feet. Light gray silt partings, 1-inch thick sand lens observed.		S-8	18-18-20	GS
					S-9		
30							

Standard Penetration Test
(140 lb. weight, 30" drop)
▲ Blows per foot

GROUNDWATER ▽

ELEVATION (feet): 60, 55, 50, 45, 40, 35

Water Content (%): 0 20 40 60 80 100

Plastic Limit |———●———| Liquid Limit
Natural Water Content

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

Edgewater Creek Bridge
Everett, Washington

BORING:
BH-2a
PAGE: 1 of 3

PROJECT NO.: 2019-157-21   FIGURE: A-8

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Sonic (4.5 inch), GeoProbe 8140 LC
SAMPLING METHOD: Sonic, SPT w/Autohammer, and D&M w/Autohammer
LOCATION: See Figure 2

DATE STARTED: 3/31/2020
DATE COMPLETED: 4/1/2020
LOGGED BY: B. Thurber
SURFACE ELEVATION: 64.0 ± feet

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
30		SC	Hard and very dense, gray, fine SAND, SILT, and CLAY, wet. Potentially interbedded, sample highly disturbed.	■	S-9a	30-31-50/5"	
				■	S-9b		AL %F
		SW SM	Very dense, gray, clean and silty, fine to coarse SAND, wet. Scattered fine gravel.		S-10		
			(WHIDBEY FORMATION)				
35			Grades to clean. Trace fine gravel broken fragments of macoma (clam) sea shells.		S-11	18-22-30	GS
			Casing sunk approx. 3 feet below bridge deck. Additional drilling required to facilitate removal of sonic sample. No SPT sample collected at 40 feet.				
40		SM	Gray, silty, fine to medium SAND, wet. Highly disturbed, mostly liquified.		S-12		
45			Scattered medium to coarse sand and fine gravel observed.		S-13		
					S-14		
50			Faintly bedded.		S-15	50/6"	
					S-16		
		SM	Very dense, dark gray, silty, fine SAND, wet. Few 1/4 inch gray clay clumps. Faintly stratified. Trace dark brown organics, approx. 1/8 inch in sampler tip. Blow counts understated due to sample disturbance by sonic coring.		S-17	8-6-8	GS
55							
			Scattered clam shell fragments, up to 3/8 inches long.				
			Casing sunk below bridge deck. Added casing to 65 feet to		S-18		
60							

Standard Penetration Test
(140 lb. weight, 30" drop)
▲ Blows per foot

GROUNDWATER

ELEVATION9(feet)

Water Content (%)

Plastic Limit ●—— Liquid Limit
Natural Water Content

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GeoSciences Inc.

Edgewater Creek Bridge
Everett, Washington

BORING:
BH-2a

PAGE: 2 of 3

PROJECT NO.: 2019-157-21   FIGURE: A-9

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Sonic (4.5 inch), GeoProbe 8140 LC
SAMPLING METHOD: Sonic, SPT w/Autohammer, and D&M w/Autohammer
LOCATION: See Figure 2

DATE STARTED: 3/31/2020
DATE COMPLETED: 4/1/2020
LOGGED BY: B. Thurber
SURFACE ELEVATION: 64.0 ± feet

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE / SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
60			retrieve the 55 to 60-foot sample. No SPT sample collected. Grades to gray. Abundant shell fragments to 63 feet. Clay layers from approx. 60.5 to 62 feet.	S-19		
		SM	Very dense, dark gray, gravelly, silty, fine to coarse SAND, wet.	S-20		
65			Grades to gray, sand fraction grades to fine, no gravel observed. No bedding, bioturbated. Scattered clam shell fragments and fine dark brown organics. Contains dark brown, woody, terrestrial organics.	S-21	30-38-45	
				S-22		
		ML	Hard, grayish brown, fine sandy, SILT, wet. No shells observed.	S-23		
70			Becomes moist to wet. Trace partially decomposed terrestrial organics. No apparent bedding.	S-24	30-38-50/6"	AL %F
		ML	Hard, dark gray, SILT, moist. Silt plastic and non-plastic.	S-25		
			Becomes gray, fine to coarse gravelly, slightly sandy. Silt plastic to non-plastic.	S-26		
75			Grades to greenish gray.	S-27	34-50/5"	
			Borehole terminated at about 76 feet below ground surface (bgs). Groundwater observed at about 8.7 feet during drilling. Borehole abandoned with 3/8" bentonite chips.			
80						
85						
90						

GROUNDWATER

Standard Penetration Test
(140 lb. weight, 30" drop)
▲ Blows per foot

Water Content (%): 0, 20, 40, 60, 80, 100

Plastic Limit ●——— Liquid Limit
Natural Water Content

ELEVATION (feet): 0, -5, -10, -15, -20, -25

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

Edgewater Creek Bridge
Everett, Washington

BORING: **BH-2a**

PAGE: 3 of 3

PROJECT NO.: 2019-157-21   FIGURE: A-10

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Mud Rotary, Dietrich D-120 Truck Rig
SAMPLING METHOD: SPT w/ Autohammer
LOCATION: See Figure 2

DATE STARTED: 3/17/2020
DATE COMPLETED: 3/17/2020
LOGGED BY: A.Mahmoud
SURFACE ELEVATION: 71.4 ± feet

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
0		ML	Lowered casing through corehole in concrete deck to ground surface, about 60 feet below. Very soft, rust-mottled yellow brown, slightly gravelly, SILT, moist. Woody debris observed. **(COLLUVIUM)**	⊠	S-0	0	
5							
		CL	Stiff, rust-banded olive-gray, CLAY, moist.	⊠	S-1	9-10-12	
		SC	Stiff, olive brown, clayey, fine SAND, moist.	⊠	S-2	10-9-4	
10							
		SM	Medium dense, olive-brown, silty, fine SAND, moist. Interbedded with silt and clay lenses.	⊠	S-3	2-10-12	
		CL	Soft, olive-gray, sandy CLAY, wet.	⊠	S-4	0-0-5	
15							
		SC	Dense, rust-mottled gray, clean, fine SAND, wet. With interbedded of clay layers. **(TRANSITIONAL BEDS)**	⊠	S-5	12-16-18	
20		SM	Dense, gray, silty and clean, fine SAND, wet. With interbedded clay layers.	⊠	S-6	14-15-21	
25			Becomes very dense.	⊠	S-7	20-27-42	
30							

Standard Penetration Test
(140 lb. weight, 30" drop)
▲ Blows per foot

Water Content (%)

Plastic Limit |———●———| Liquid Limit
Natural Water Content

ELEVATION (feet): 70, 65, 60, 55, 50, 45

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

Edgewater Creek Bridge
Everett, Washington

BORING:
BH-3

PAGE: 1 of 2

PROJECT NO.: 2019-157-21   FIGURE: A-11

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Mud Rotary, Dietrich D-120 Truck Rig
SAMPLING METHOD: SPT w/ Autohammer
LOCATION: See Figure 2

DATE STARTED: 3/17/2020
DATE COMPLETED: 3/17/2020
LOGGED BY: A.Mahmoud
SURFACE ELEVATION: 71.4 ± feet

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
30				⊠	S-8	30-50/6"		
35				⊠	S-9	20-27-42		
40				⊠	S-10	36-50/6"		

Borehole terminated at 41 feet due to loss of sampler sub
fitting downhole, blocking further drilling.
Borehole abandoned with bentonite chips.

Standard Penetration Test
(140 lb. weight, 30" drop)
▲ Blows per foot

SPT plot axis: 0, 10, 20, 30, 40, 50

ELEVATION (feet): 40, 35, 30, 25, 20, 15

Water Content (%): 0, 20, 40, 60, 80, 100

Plastic Limit ●—— Liquid Limit
Natural Water Content

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated
and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

Edgewater Creek Bridge
Everett, Washington

BORING:
BH-3

PAGE: 2 of 2

PROJECT NO.: 2019-157-21   FIGURE: A-12

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Sonic (4.5 inch), GeoProbe 8140 LC
SAMPLING METHOD: Sonic, SPT w/Autohammer, and D&M w/Autohammer
LOCATION: See Figure 2

DATE STARTED: 4/1/2020
DATE COMPLETED: 4/3/2020
LOGGED BY: B. Thurber
SURFACE ELEVATION: 62.0 ± feet

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE / SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
0		ML	Lowered casing through corehole in concrete deck to ground surface, about 60 feet below. Bridge deck 3 inches A.C.P. over 8 inches concrete. Casing pushed to 5 feet, sonic cored 0 to 5 feet.	S-1		
			Very soft, mixed bluish gray and brown, slightly gravelly, slightly sandy, clayey SILT, moist. Scattered organics. **(COLLUVIUM)**	S-2		
5		CL	At 3.5 ft becomes rust-mottled light olive brown; with woody debris. Chunks of clay/silt from 4.5 to 5 feet.	S-3a S-3b	13-13-14	AL %F
		ML	Hard, olive brown grading to olive gray, sandy, lean CLAY, moist.	S-3c		
		ML	Very dense and hard, rust-banded olive gray to olive brown, fine SAND, SILT and CLAY, moist. Finely bedded at approx. 15 degrees.	S-4		
			Gray, SILT, moist. Fine bedding at 10 to 30 degrees.			
10		ML	Soft, rust-banded light olive brown, fine sandy, SILT, moist.	S-5	0-2-2	
		CL ML	Hard, rust-banded olive brown, sandy, silty, CLAY, moist. Finely interbedded with silty to clean, fine SAND, wet, up to 1 inch thick. **(TRANSITIONAL BEDS)**	S-6		
15			Sand fraction greenish gray.	S-7	10-15-31	AL %F
		SP	Gray, clean, fine to medium SAND, wet. Interbedded with greenish gray and gray clay layers from 19 to 20 feet, approx. 3/8 inches thick.	S-8		
20		ML	Hard, grayish brown, very sandy, SILT, wet. With greenish-gray clay layers up to 3/8 inches thick. Appox. 20 to 25% clay layers.	S-9a S-9b	16-26-36	
				S-10		
25		SM	Dense, dark gray, very silty, SAND, wet. Sample highly disturbed, looks slightly liquified. Blow counts understated due to sample disturbance by sonic coring. Sample coarsens upward. Two layers of clean, fine to medium sand from 26.5 to 30 feet, approx. 3 inches thick. **(WHIDBEY FORMATION)**	S-11	4-9-18	
				S-12		
30						

Standard Penetration Test
(140 lb. weight, 30" drop)
▲ Blows per foot

Plastic Limit ●— Liquid Limit
Natural Water Content
Water Content (%)

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GeoSciences Inc.

Edgewater Creek Bridge
Everett, Washington

BORING:
BH-3a

PAGE: 1 of 3

PROJECT NO.: 2019-157-21   FIGURE: A-13

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Sonic (4.5 inch), GeoProbe 8140 LC
SAMPLING METHOD: Sonic, SPT w/Autohammer, and D&M w/Autohammer
LOCATION: See Figure 2

DATE STARTED: 4/1/2020
DATE COMPLETED: 4/3/2020
LOGGED BY: B. Thurber
SURFACE ELEVATION: 62.0 ± feet

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE / SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	ELEVATION (feet)
30			Becomes fine to medium sand, with layers of silty fine sand and one 2-inch layer of greenish gray clay, moist. Blow counts understated due to sample disturbance by sonic coring.	S-13	9-18-23		
		SM	Very dense, dark olive gray, silty fine SAND, wet. Few greenish-gray clay layers observed.	S-14 S-15			40
35		SM	Very dense, gray and dark gray, silty, fine SAND and clean, fine to medium SAND, wet. Stratified. Minor to few clay lenses throughout, ranging in thickness from 1/4 inch to 1 inch. Clay lenses in sampler tip. Blow counts understated due to sample disturbance by sonic coring. Grades to fine sand. Few greenish-gray clay layers, up to 1/2 inch thick.	S-16 S-17	0-8-18		35
40			Blow counts understated due to sample disturbance by sonic coring.	S-18	0-10-15	GS	30
			Few clay lenses, approx. 1/4 inches thick.	S-19			
45			Becomes micaceous. Scattered clay layers, approx. 1/4 to 1/2 inches thick.	S-20			25
				S-21			
50			Grades to olive-gray and gray. Minor clay clumps observed. Blow counts understated due to sample disturbance by sonic coring.	S-22	3-6-10		20
		SM	Very dense, dark gray, slightly silty, fine SAND, wet. Few dark brown organics and trace bivalve shell fragments. Trace clay layers, approx. 1/4 inch thick.	S-23		GS	
55			2 clay layers, approx. 1/4 inch thick each.	S-24			15
		SM	Gray to dark gray, gravelly, silty, fine to medium SAND, wet. Gravel fraction fine to coarse. Abundant shell fragments observed. One clay lens observed, approx. 1 inch thick.				
60							

Standard Penetration Test
(140 lb. weight, 30" drop)
▲ Blows per foot

Water Content (%)

Plastic Limit ●—— Liquid Limit
Natural Water Content

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

Edgewater Creek Bridge
Everett, Washington

BORING:
BH-3a

PAGE: 2 of 3

PROJECT NO.: 2019-157-21   FIGURE: A-14

BORING-DSM 2019-157-21.GPJ 6/24/20

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Sonic (4.5 inch), GeoProbe 8140 LC
SAMPLING METHOD: Sonic, SPT w/Autohammer, and D&M w/Autohammer
LOCATION: See Figure 2

DATE STARTED: 4/1/2020
DATE COMPLETED: 4/3/2020
LOGGED BY: B. Thurber
SURFACE ELEVATION: 62.0 ± feet

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE / SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
60		SM	Very dense, dark gray, silty, fine to medium SAND, wet.	S-25a / S-25b	10-50/5"	
		GP GM	Olive gray, slightly silty, fine to coarse sandy, fine GRAVEL, wet.			
		SW SM	Very dense, gray, slightly silty, slightly fine to coarse gravelly, fine to coarse SAND. wet. Sand grades to fine to medium, gravel to fine. Trace shell fragments.	S-26		
65				S-27	30-50/5"	
		SM	Olive gray. silty fine SAND, moist. Trace shell fragments and fine brown organics. Massive, bioturbated.	S-28 S-29		
70		SM	Very dense, dark grayish-brown, very silty, fine SAND, moist. Finely bedded with trace dark brown, fibrous, terrestrial organics.	S-30	10-31-36	%F
		ML	Dark gray, SILT, moist. Observed burrows filled with fine sand. No apparent bedding.	S-31		
		GM	Very dense, gray, silty, fine to coarse sandy, fine to coarse GRAVEL, moist to wet.	S-32 S-33	50/3"	
75						

Borehole terminated at about 75.25 feet below ground surface (bgs).
Groundwater observed at about 11 feet when casing at 20 feet.
Borehole abandoned with 3/8" bentonite chips.

GROUNDWATER

Standard Penetration Test
(140 lb. weight, 30" drop)
▲ Blows per foot

ELEVATION (feet)

Plastic Limit ├──●──┤ Liquid Limit
Natural Water Content

HWA GeoSciences Inc.

Edgewater Creek Bridge
Everett, Washington

BORING:
BH-3a

PAGE: 3 of 3

PROJECT NO.: 2019-157-21   FIGURE: A-15

BORING-DSM 2019-157-21.GPJ 6/24/20

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Mud Rotary, Dietrich D-120 Truck Rig
SAMPLING METHOD: SPT w/ Autohammer
LOCATION: See Figure 2

DATE STARTED: 3/16/2020
DATE COMPLETED: 3/17/2020
LOGGED BY: A. Mahmoud
SURFACE ELEVATION: 121.2 ± feet

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
0		SM	Asphalt concrete pavement.		S-1	15-25-38	GS	
			Very dense, dark yellowish brown, gravelly, silty, SAND, moist. **(FILL)**		S-2	8-7-6		
5		SM	Medium dense, brown, silty, gravelly, SAND, moist.		S-3	6-4-6		
			No recovery.		S-4	2-3-4		
10		CL	Stiff, light olive brown, CLAY, wet. **(COLLUVIUM)**		S-5	3-4-7		
		CL	Very stiff, light olive brown, CLAY, wet. **(TRANSITIONAL BEDS)**		S-6	20-26-30		
15			Very stiff, rust-mottled gray grading to yellow brown, lean CLAY, wet. Blocky nature and laminated.		S-7	15-10-14		
					S-8	9-10-15		
20		CH	Very stiff, gray, fat CLAY, wet.		S-9	9-10-14	AL %F	
25		CL	Very stiff, gray, CLAY, wet.		S-10	11-7-25		
30								

Standard Penetration Test
(140 lb. weight, 30" drop)
▲ Blows per foot

ELEVATION (feet)

0   10   20   30   40   50

Water Content (%)
0   20   40   60   80   100

Plastic Limit |——●——| Liquid Limit
Natural Water Content

HWA GEOSCIENCES INC.

Edgewater Creek Bridge
Everett, Washington

BORING: BH-4

PAGE: 1 of 4

PROJECT NO.: 2019-157-21   FIGURE: A-16

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Mud Rotary, Dietrich D-120 Truck Rig
SAMPLING METHOD: SPT w/ Autohammer
LOCATION: See Figure 2

DATE STARTED: 3/16/2020
DATE COMPLETED: 3/17/2020
LOGGED BY: A. Mahmoud
SURFACE ELEVATION: 121.2 ± feet

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
30					S-11	14-24-25	
35		ML	Very stiff, grayish brown, sandy, SILT, moist.		S-12	27-39-44	GS
40			Clay lenses interbedded.		S-13	44-50/6"	
45					S-14	25-50/6"	GS
50		CL	Very stiff, rust-mottled grayish brown, lean CLAY, moist. Interbedded with sand. Blocky structure.		S-15	20-35-45	
55					S-16	18-39-50/6"	
60							

Standard Penetration Test
(140 lb. weight, 30" drop)
▲ Blows per foot

Water Content (%)
Plastic Limit ●—— Liquid Limit
Natural Water Content

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GeoSciences Inc.

Edgewater Creek Bridge
Everett, Washington

BORING:
BH-4

PAGE: 2 of 4

PROJECT NO.: 2019-157-21   FIGURE: A-17

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Mud Rotary, Dietrich D-120 Truck Rig
SAMPLING METHOD: SPT w/ Autohammer
LOCATION: See Figure 2

DATE STARTED: 3/16/2020
DATE COMPLETED: 3/17/2020
LOGGED BY: A. Mahmoud
SURFACE ELEVATION: 121.2 ± feet

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
60			Becomes gray. Blocky structure.		S-17	22-32-50/5"	AL
65					S-18	50/5"	
70					S-19	25-50/6"	
75			Becomes grayish-brown, rust banded.		S-20	20-40-44	
80		SM	Very dense, rust-banded dark gray, silty, fine SAND, moist. Clay lenses interbedded. **(WHIDBEY FORMATION)**		S-21	18-23-24	
85					S-22	18-23-28	
90							

Standard Penetration Test
(140 lb. weight, 30" drop)
▲ Blows per foot

ELEVATION (feet)

Water Content (%)

Plastic Limit ├──●──┤ Liquid Limit
Natural Water Content

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated
and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

Edgewater Creek Bridge
Everett, Washington

BORING:
BH-4

PAGE: 3 of 4

PROJECT NO.: 2019-157-21   FIGURE: A-18

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Mud Rotary, Dietrich D-120 Truck Rig
SAMPLING METHOD: SPT w/ Autohammer
LOCATION: See Figure 2

DATE STARTED: 3/16/2020
DATE COMPLETED: 3/17/2020
LOGGED BY: A. Mahmoud
SURFACE ELEVATION: 121.2 ± feet

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
90				⊠	S-23	16-25-29		
95				⊠	S-24	21-29-32		
100				⊠	S-25	33-45-50/6"		

Borehole terminated at 101.5 feet below ground surface (bgs).
Borehole abandoned with 3/8-inch bentonite chips.

Standard Penetration Test
(140 lb. weight, 30" drop)
▲ Blows per foot

Water Content (%)

Plastic Limit ●————● Liquid Limit
Natural Water Content

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

Edgewater Creek Bridge
Everett, Washington

BORING:
BH-4

PAGE: 4 of 4

PROJECT NO.: 2019-157-21

FIGURE: A-19

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Sonic (4.5 inch), GeoProbe 8140 LC
SAMPLING METHOD: Sonic, SPT w/Autohammer
LOCATION: See Figure 2

DATE STARTED: 4/3/2020
DATE COMPLETED: 4/3/2020
LOGGED BY: B. Thurber
SURFACE ELEVATION: 121.0 ± feet

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE / SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
0		SM	4 inches A.C.P.			
			Medium dense, olive-brown, silty, gravelly, fine to coarse SAND, moist.	S-1		
			(FILL)			
		MH	4 inches Concrete (old roadway surface)			
5			Medium stiff, rust-mottled, banded yellow-brown and light olive-brown, elastic SILT, moist.	S-2	3-6-9	AL %F
			(COLLUVIUM)			
			Tan silt beds broken up. Minor sand lenses.	S-3		
			Observed tan silt laminae. Bedding near horizontal. Some rust banding.	S-4		
			Fine, blocky texture.			
10		ML	Hard, gray, SILT, moist, laminated.	S-5	6-18-20	
			(TRANSITIONAL BEDS)			
			Observed light gray silt partings. Bedding sub-horizontal, disturbed.	S-6a / S-6b / S-6c	15-22-34	
15		ML ML	Hard, rust-banded olive brown, SILT, CLAY, and SAND, moist. Finely bedded.	S-7 / S-8	6-10-15	
			Hard, gray, SILT, moist. Laminated, with light gray silt partings. Bedding sub-horizontal, approx. 10 degrees. Becomes laminated to finely bedded, up to 1/4 inch thick. Broken up from approx. 17 to 18 feet due to high amount of silt partings.	S-9		
				S-10		
20			Fine silt partings, about 1 to 3 inches apart.	S-11	6-10-15	
			Bedding about 20 degrees, some beds 1/4 inch thick. Breaks along major silt partings, a few inches apart.	S-12		
25			Few silt partings. Bedding approx. horizontal.	S-13	10-15-25	
			Few fine sand partings.	S-14		
30			Coarse silt partings.	S-15	16-30-36	
			Borehole terminated at about 31.5 feet below ground surface (bgs). No groundwater observed during drilling. Borehole abandoned with 3/8" bentonite chips.			
35						

Standard Penetration Test
(140 lb. weight, 30" drop)
▲ Blows per foot

Water Content (%)
Plastic Limit |——●——| Liquid Limit
Natural Water Content

ELEVATION (feet)

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

Edgewater Creek Bridge
Everett, Washington

BORING:
BH-5
PAGE: 1 of 1

PROJECT NO.: 2019-157-21   FIGURE: A-20

DRILLING COMPANY: Geologic Drill Partners
DRILLING METHOD: Hollow Stem Auger, Acker Soil Mechanic
SAMPLING METHOD: SPT w/Cathead
LOCATION: See Figure 2

DATE STARTED: 3/30/2020
DATE COMPLETED: 3/30/2020
LOGGED BY: B. Thurber
SURFACE ELEVATION: 121.0 ± feet

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE / SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
0			Ground surface vegetated with brush and understory plants. Loose, dark brown, silty SAND and sandy SILT, moist. **(TOPSOIL)**			
		SM	Loose, rust-mottled olive brown and olive gray, very silty SAND, moist. With 2 inch bed of clay and 1/4 inch layer of wood. **(COLLUVIUM)**	S-1	2-2-6	
		ML	1/2 inch organic-rich layer.			
5			Very stiff, light olive-brown, SILT, moist. Finely bedded to laminated, with silt partings. Minor rust banding. **(TRANSITIONAL BEDS)**	S-2	6-9-14	
		ML	Hard, gray, SILT, moist. Laminated.	S-3	9-14-17	AL %F
			Rust-covered joints, approx. 40 and 45 degrees. No slickensides.			
10				S-4	9-15-25	
			Approx. 2-inch-thick, rust-mottled olive brown laminae. Laminae at approx. 50 degrees.			
				S-5	9-19-31	
			High angle, rusted joints, approx. 85 to 90 degrees. Rough, no slickensides.	S-6a	19-50/5"	
15			Grades to olive gray. Fine sand lens with flame structure.	S-6b		
			Borehole terminated at about 15 feet below ground surface (bgs). No groundwater observed during drilling. Borehole abandoned with 3/8" bentonite chips.			
20						

Standard Penetration Test
(140 lb. weight, 30" drop)
▲ Blows per foot

Water Content (%)
Plastic Limit ⊢—●—⊣ Liquid Limit
Natural Water Content

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

Edgewater Creek Bridge
Everett, Washington

HWA GeoSciences Inc.

BORING: **BH-6**
PAGE: 1 of 1

PROJECT NO.: 2019-157-21   FIGURE: A-21

DRILLING COMPANY: Geologic Drill Partners
DRILLING METHOD: Hollow Stem Auger, Acker Soil Mechanic
SAMPLING METHOD: SPT w/Cathead
LOCATION: See Figure 2

DATE STARTED: 3/30/2020
DATE COMPLETED: 3/30/2020
LOGGED BY: B. Thurber
SURFACE ELEVATION: 121.2 ± feet

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
0			Loose, brown TOPSOIL.					
		ML	Medium stiff, olive-brown, gravelly, very sandy, SILT, moist. **(COLLUVIUM)**					
					S-1a	3-5-3		
		ML	Approx. 1 inch fine asphalt debris. Medium stiff, rust-banded olive brown, SILT, moist. Laminated.		S-1b			
5			**(TRANSITIONAL BEDS)** Becomes stiff, no rust banding. Contains silt partings. Both finely bedded and laminated. Trace roots observed.		S-2a S-2b	3-4-7		
		ML	Medium stiff, brown,SILT, moist. Fine blocky texture, fracture zone.					
		ML	Medium stiff, olive-gray and finely rust-banded olive-brown, slightly sandy, SILT, moist. Laminated. Trace roots.		S-3	3-3-5		
10		ML	Very stiff, gray, SILT, moist, laminated, with silt partings. Bedding approx. 5 degrees.		S-4	9-13-16		
			Becomes hard.		S-5	12-20-30		
15			Borehole terminated at about 14 feet below ground surface (bgs). No groundwater was observed during drilling. Borehole abandoned with 3/8" bentonite chips.					
20								

Standard Penetration Test
(140 lb. weight, 30" drop)
▲ Blows per foot

ELEVATION (feet)

Water Content (%)

Plastic Limit ⊢——●——⊣ Liquid Limit
Natural Water Content

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

Edgewater Creek Bridge
Everett, Washington

BORING: BH-7

PAGE: 1 of 1

DRILLING COMPANY: Geologic Drill Partners
DRILLING METHOD: Hollow Stem Auger, Acker Soil Mechanic
SAMPLING METHOD: SPT w/Cathead
LOCATION: See Figure 2

DATE STARTED: 10/29/2020
DATE COMPLETED: 10/29/2020
LOGGED BY: S. Schlitt

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
0			Loose, brown, TOPSOIL. Heavily vegetated, brush and trees, roots up to 1 inch diameter.					
		ML	Soft, grayish-brown, gravelly, sandy, SILT, moist. Abudnant organics, observed mixing with topsoil material, minimal recovery. **(COLLUVIUM)**		S-1	1-2-1		
5			Becomes very sandy, wet. Observed trace gravel, organics, and rootlets.		S-2	1-1-1		▽
			Becomes medium stiff, blue-gray with brown streaks. Observed wood debris in contact. Blow counts in bottom 6 inches overstated due to wood.		S-3A	2-4-15		
					S-3B			
			Wood debris. Likely felled trees underlying landslide material. **(WOOD)**					
10			Refusal of augers at 10 feet. Continuous sampling to depth.		S-4	16-8-8		
		SM	Medium dense, dark gray, silty, SAND, wet. Trace gravel. Moisture overstated due to added water. Blow counts overstated due to sidewall friction produced by continuous sampling.		S-5	7-9-10		
			(TRANSITIONAL BEDS) Sand content becomes coarser. Observed lenses of clean sand and very silt sand, minor wood debris.		S-6	7-7-10-14		
15			Borehole terminated at about 15 feet below ground surface (bgs). Groundwater was observed at 5.3 feet bgs upon completion of drilling. Borehole abandoned with 3/8" bentonite chips.					
20								

Standard Penetration Test
(140 lb. weight, 30" drop)
▲ Blows per foot

ELEVATION (feet)

Water Content (%)

Plastic Limit ●—— Liquid Limit
Natural Water Content

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GeoSciences Inc.

Edgewater Creek Bridge
Everett, Washington

BORING:
BH-08
PAGE: 1 of 1

PROJECT NO.: 2019-157-21   FIGURE: A-23

DRILLING COMPANY: Geologic Drill Partners
DRILLING METHOD: Hollow Stem Auger, Acker Soil Mechanic
SAMPLING METHOD: SPT w/Cathead
LOCATION: See Figure 2

DATE STARTED: 10/29/2020
DATE COMPLETED: 10/29/2020
LOGGED BY: S. Schlitt

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE / SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
0			Loose, brown, TOPSOIL. Heavily vegetated, brush and trees, roots up to 1 inch diameter.				
		ML	Very soft to soft, grayish-brown, SILT, moist. Trace organics, minor send lenses, high plasticity. **(COLLUVIUM)**				▽
				S-1	1-1-1		
5			Becomes light gray.	S-2	1-2-1		
			Becomes wet, dark gray, moisture observed in sand lenses.	S-3	1-1-1		
10				S-4A	1-2-3		
		SM	Loose, dark gray, silty, SAND, wet. Trace organics, gravel. Observed minor artesian conditions, water level rising in auger upon entering sand soils. **(TRANSITIONAL BEDS)**	S-4B			
			Observed lenses of clean sand.	S-5	2-6-10		
15			Sand content becomes coarser. Observed less silt.	S-6	5-7-10		
			Borehole terminated at about 16.5 feet below ground surface (bgs). Groundwater was observed at 1.9 feet bgs upon completion of drilling. Borehole abandoned with 3/8" bentonite chips.				
20							

Standard Penetration Test
(140 lb. weight, 30" drop)
▲ Blows per foot

Water Content (%)

Plastic Limit ●—— Liquid Limit
Natural Water Content

ELEVATION (feet)

HWA GeoSciences Inc.

Edgewater Creek Bridge
Everett, Washington

BORING:
BH-09

PAGE: 1 of 1

PROJECT NO.: 2019-157-21   FIGURE: A-24

BORING-DSM 2019-157-21.GPJ 10/23/20

DRILLING COMPANY: Geologic Drill Partners
DRILLING METHOD: Hollow Stem Auger, Acker Soil Mechanic
SAMPLING METHOD: SPT w/Cathead
LOCATION: See Figure 2

DATE STARTED: 10/30/2020
DATE COMPLETED: 10/30/2020
LOGGED BY: S. Schlitt

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
0		SM	Loose, brown, TOPSOIL. Heavily vegetated, brush and trees, roots up to 1 inch diameter.				
			Medium dense, dark yellowish-brown, silty, gravelly, SAND, moist. Travel 1-2 inch gravels in cuttings. **(TRANSITIONAL BEDS)**				
					S-1	11-10-9	
5		ML	Hard, brown, SILT, moist. Trace sand. High plasticity and well consolidated. Minor lamination observed.		S-2	9-13-22	
			Becomes dark gray.		S-3	11-22-30	
10			Borehole terminated at about 9 feet below ground surface (bgs). No groundwater was observed during drilling. Borehole abandoned with 3/8" bentonite chips.				
15							
20							

Standard Penetration Test
(140 lb. weight, 30" drop)
▲ Blows per foot

GROUNDWATER

ELEVATION (feet)

Water Content (%)

Plastic Limit ├──●──┤ Liquid Limit
Natural Water Content

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GeoSciences Inc.

Edgewater Creek Bridge
Everett, Washington

BORING:
BH-10

PAGE: 1 of 1

PROJECT NO.: 2019-157-21   FIGURE: A-25

BORING-DSM 2019-157-21.GPJ 10/23/20

DRILLING COMPANY: Geologic Drill Partners
DRILLING METHOD: Hollow Stem Auger, Acker Soil Mechanic
SAMPLING METHOD: SPT w/Cathead
LOCATION: See Figure 2

DATE STARTED: 10/30/2020
DATE COMPLETED: 10/30/2020
LOGGED BY: S. Schlitt

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
0		ML	Loose, brown, TOPSOIL. Heavily vegetated, brush and trees, roots up to 1 inch diameter.					
			Very stiff, light brownish-gray, SILT, moist. Observed minor rootlets, organics, and fine gravel in cuttings. Well consolidated and thinly laminated. **(TRANSITIONAL BEDS)**		S-1	6-11-10		
5		CL	Hard, gray-brown, CLAY, moist. High plasticity, well consolidated, and laminated		S-2	9-13-24		
		ML	Hard, light brownish-gray, SILT, moist. Well consolidated and laminated.		S-3	16-25-32		
10			Borehole terminated at about 9 feet below ground surface (bgs). No groundwater was observed during drilling. Borehole abandoned with 3/8" bentonite chips.					
15								
20								

Standard Penetration Test
(140 lb. weight, 30" drop)
▲ Blows per foot

Water Content (%)

Plastic Limit ⊢——●——⊣ Liquid Limit
Natural Water Content

HWA GeoSciences Inc.

Edgewater Creek Bridge
Everett, Washington

BORING: BH-11
PAGE: 1 of 1

PROJECT NO.: 2019-157-21   FIGURE: A-26

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Hollow Stem Auger, Dietrich D-50 Track Rig
SAMPLING METHOD: SPT w/Autohammer
LOCATION: See Figure 2

DATE STARTED: 7/16/2021
DATE COMPLETED: 7/16/2021
LOGGED BY: M.A. Benson

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE / SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
0		SM	Grass on surface.			
		CL	Dark brown, silty, SAND, moist. Abundant roots. **(TOPSOIL)**			
			Stiff, olive brown grading to olive gray, sandy, CLAY, moist. Trace gravel, low plasticity. **(FILL)**	S-1	3-3-5	
5		SM	Very dense, brown, silty, gravelly, SAND, moist. Drove rock, low recovery. Blow counts likely overstated.	S-2	8-25-26	GS
		MH	Very stiff, rust-mottled olive brown, elastic SILT, moist. Mostly massive, with occassional bedding. Dark streaks on bedding. **(TRANSITIONAL BEDS)**	S-3	5-7-10	
10			Grades to rust-mottled olive gray to gray.	S-4	5-10-10	
			Very stiff, dark gray, elastic SILT, moist. Bedded.	S-5	5-10-14	
15			Fine sand on some bedding planes.	S-6	5-10-13	AL %F
20		CL	Very stiff, dark gray, CLAY, moist.	S-7	8-8-15	
25				S-8	7-13-14	
			Chattery drill action.			
30						

GROUNDWATER

Standard Penetration Test
(140 lb. weight, 30" drop)
▲ Blows per foot

ELEVATION (feet)

Water Content (%)

Plastic Limit ⊢●⊣ Liquid Limit
Natural Water Content

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GeoSciences Inc.

Edgewater Creek Bridge
Everett, Washington

BORING:
BH-12

PAGE: 1 of 2

PROJECT NO.: 2019-157-21   FIGURE: A-27

BORING-DSM 2019-157-21.GPJ 7/28/21

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Hollow Stem Auger, Dietrich D-50 Track Rig
SAMPLING METHOD: SPT w/Autohammer
LOCATION: See Figure 2

DATE STARTED: 7/16/2021
DATE COMPLETED: 7/16/2021
LOGGED BY: M.A. Benson

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
30		CL	Becomes hard.		S-9	8-15-20	
35			Hard, dark gray, sandy, CLAY, moist. Sand interbeds.		S-10	8-21-27	
40			No sand interbeds, oxidized to olive brown from about 40 to 40.4 feet.		S-11	14-22-26	
45		SP SM	Very dense, gray, slightly silty, fine SAND, moist. **(WHIDBEY FORMATION)** Chattery drill action.		S-12	29-45-50/5"	
50		SP	Very dense, gray, silty, fine SAND, moist. Finely bedded.		S-13	18-36-50/6"	

BH-12 terminated at about 51.5 feet below ground surface.
No groundwater observed during drilling.
Borehole abandoned with 3/8" bentonite chips.

Standard Penetration Test
(140 lb. weight, 30" drop)
▲ Blows per foot

Plastic Limit ●——● Liquid Limit
Natural Water Content

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated
and therefore may not necessarily be indicative of other times and/or locations.

HWA GeoSciences Inc.

Edgewater Creek Bridge
Everett, Washington

BORING:
BH-12

PAGE: 2 of 2

PROJECT NO.: 2019-157-21   FIGURE: A-28

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Hollow Stem Auger, Dietrich D-50 Track Rig
SAMPLING METHOD: SPT w/Autohammer
LOCATION: See Figure 2

DATE STARTED: 7/16/2021
DATE COMPLETED: 7/16/2021
LOGGED BY: M.A. Benson

Log Columns

DEPTH (feet) | SYMBOL | USCS SOIL CLASS | DESCRIPTION | SAMPLE TYPE | SAMPLE NUMBER | PEN. RESISTANCE (blows/6 inches) | OTHER TESTS | GROUNDWATER | ELEVATION (feet)

Standard Penetration Test
(140 lb. weight, 30" drop)
▲ Blows per foot

Depth	USCS	Description	Sample No.	Pen. Resistance
0	SM / SM	Grass on surface, about 6 inches of dark brown, silty, SAND, moist. Abundant organics. **(TOPSOIL)**		
		Medium dense, Rust-mottled olive brown, very silty, fine SAND, moist. Finely bedded. Coarsens up. **(FILL)**	S-1	6-9-15
5	ML	Stiff, rust-mottled olive brown, sandy, SILT, moist. Bedded, coarsens up.	S-2	3-7-7
	SM	Medium dense, brown, silty, SAND, moist. Abundant rootlets and organics.	S-3a	3-3-11
	CL	Stiff, rust-mottled gray brown, CLAY, moist. Bedded. **(TRANSITIONAL BEDS)**	S-3b	
10		Becomes gray.	S-4	5-8-11
	MH	Hard, dark gray, elastic SILT, moist. Bedded, low plasticity. Siltstone gravel in sampler shoe, blow counts likely overstated. Fine sand bedding.	S-5	8-15-17
15		Becomes very stiff no bedding or gravel.	S-6	6-8-12
20			S-7	6-10-13
		Finely bedded.		
25	CL	Very stiff, gray, CLAY, moist.	S-8	11-15-18
30				

Water Content (%): 0 — 20 — 40 — 60 — 80 — 100

Plastic Limit ●—— Liquid Limit
Natural Water Content

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GeoSciences Inc.

Edgewater Creek Bridge
Everett, Washington

BORING: **BH-13**
PAGE: 1 of 2

PROJECT NO.: 2019-157-21   FIGURE: A-29

BORING-DSM 2019-157-21.GPJ 7/28/21

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Hollow Stem Auger, Dietrich D-50 Track Rig
SAMPLING METHOD: SPT w/Autohammer
LOCATION: See Figure 2

DATE STARTED: 7/16/2021
DATE COMPLETED: 7/16/2021
LOGGED BY: M.A. Benson

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
30		CL	Becomes hard, bedded to finely bedded. Few fine sand beds.		S-9	10-15-18	AL %F
35			Hard, dark gray, sandy, CLAY, moist. Contains grayish-brown sand beds with oxidation on bedding planes.		S-10	12-19-28	
			Chattery drill action.				
40		SC	Very dense, rust-mottled olive brown, very clayey, fine SAND, moist. Contains clay interbeds. **(WHIDBEY FORMATION)**		S-11	11-32-40	
45		SM	Very dense, Gray, silty, fine SAND, moist. Bedded, few silt interbeds.		S-12	28-36-40	GS

BH-13 terminated at about 46.5 feet below ground surface.
No groundwater observed during drilling.
Borehole abandoned with 3/8" bentonite chips.

Standard Penetration Test
(140 lb. weight, 30" drop)
▲ Blows per foot

Plastic Limit ⊢●⊣ Liquid Limit
Natural Water Content

GROUNDWATER

ELEVATION (feet)

Water Content (%)

HWA GeoSciences Inc.

Edgewater Creek Bridge
Everett, Washington

BORING:
BH-13
PAGE: 2 of 2

PROJECT NO.: 2019-157-21   FIGURE: A-30

BORING-DSM 2019-157-21.GPJ 7/28/21

DRILLING COMPANY: HWA GeoSciences Inc.
DRILLING METHOD: Hand Bucket Auger
SAMPLING METHOD: Grab
LOCATION: See Figure 2

SURFACE ELEVATION: 124.50 ± feet
CASING ELEVATION ± feet

DATE STARTED: 3/5/2020
DATE COMPLETED: 3/5/2020
LOGGED BY: B. Thurber

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION
0		ML	Loose to soft, brown grading to light brown, sandy SILT, moist. Scattered gravel and partly decayed wood. **(COLLUVIUM)**
		CL	Very stiff, gray, CLAY, moist. Clumps of tan with rust mottling below 3ft.
		SM	Dense, yellow-brown, fine to medium SAND with silt beds, moist to damp..
5		ML	Very stiff to hard, olive-brown, SILT, moist. Thin lenses of sand.
		CL	**(TRANSITIONAL BEDS)** Very stiff to hard, olive-brown, CLAY, moist, laminated with gravel dropstones.
			Handhole and DCP terminated at 7.7 feet. Minor groundwater seepage encountered at approx. 7.5 feet during excavation.
10			

SAMPLE TYPE
SAMPLE NUMBER
PEN. RESISTANCE (blows/6 inches)
OTHER TESTS
GROUNDWATER

▲ Non-Standard Penetration Resistance (Blows per foot)
(17.6 lb. weight, 22.6" drop)

Water Content (%)

Plastic Limit ├──●──┤ Liquid Limit
Natural Water Content

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GeoSciences Inc.

Edgewater Creek Bridge
Everett, Washington

HAND HOLE:
HH-1

PAGE: 1 of 1

DRILLING COMPANY: HWA GeoSciences Inc.
DRILLING METHOD: Hand Bucket Auger
SAMPLING METHOD: Grab
LOCATION: See Figure 2

SURFACE ELEVATION: 63.70 ± feet
CASING ELEVATION ± feet

DATE STARTED: 3/5/2020
DATE COMPLETED: 3/5/2020
LOGGED BY: B. Thurber

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION
0		ML	Loose to soft, light olive brown, slightly gravelly SILT, moist. Partly decomposed wood with 2-3" diameter clumps of laminated gray clay below 1.75'. **(COLLUVIUM)**
		SM	Medium dense, yellow-brown, slightly gravelly silty SAND, moist. Charcoal at 3.75'.
5		ML	Very stiff, rust-mottled brown, SILT, moist. With thin lenses of sand with gravel. **(WHIDBEY FORMATION)**
			Handhole and DCP terminated at 7.5 feet. No groundwater seepage observed during excavation.
10			

▲ Non-Standard Penetration Resistance (Blows per foot)
(17.6 lb. weight, 22.6" drop)

Plastic Limit ⊢●⊣ Liquid Limit
Natural Water Content

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GeoSciences Inc.

Edgewater Creek Bridge
Everett, Washington

HAND HOLE:
HH-2

PAGE: 1 of 1

PROJECT NO.: 2019-157-21   FIGURE: **A-32**
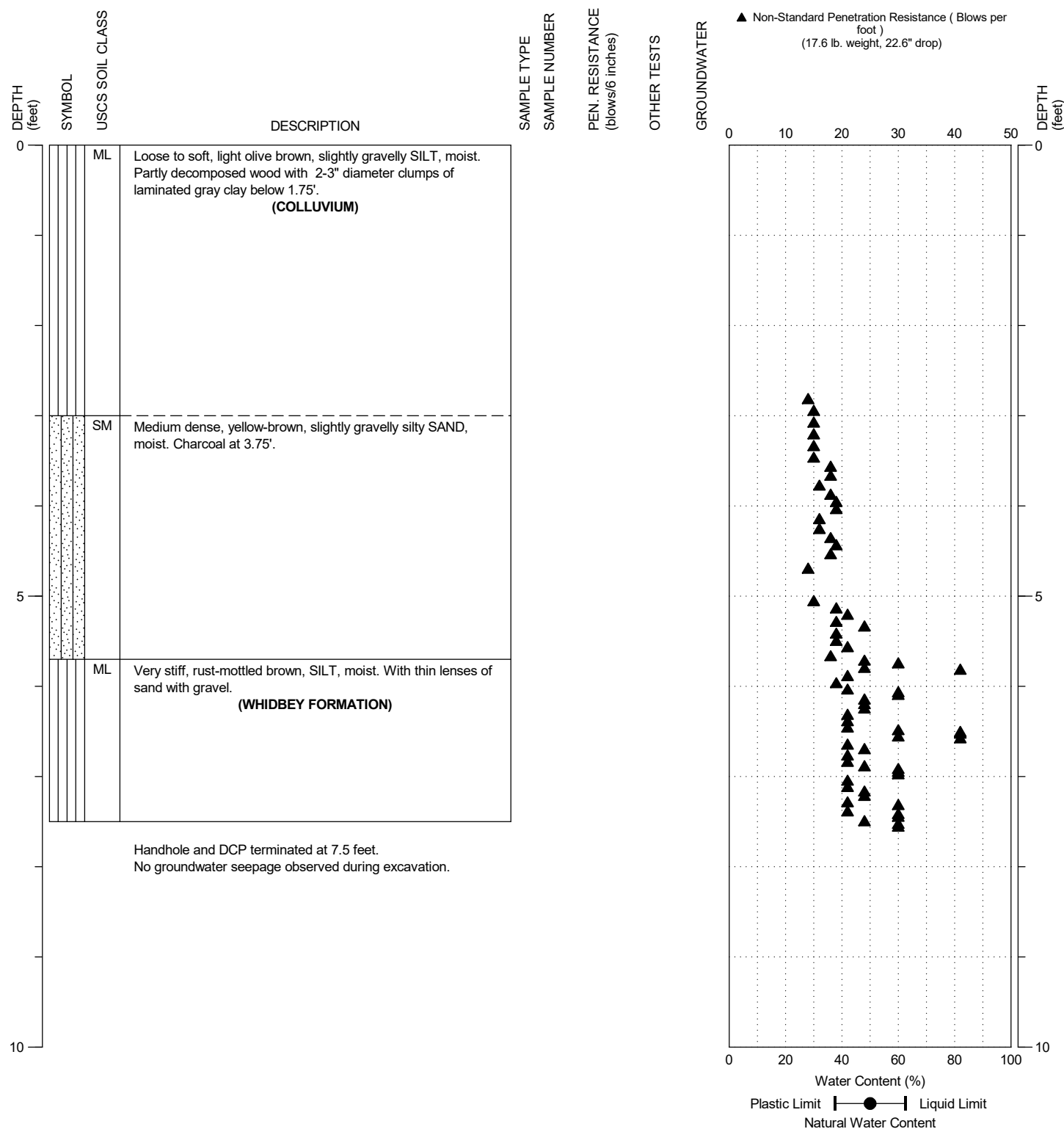
DRILLING COMPANY: HWA GeoSciences Inc.
DRILLING METHOD: Hand Bucket Auger
SAMPLING METHOD: Grab
LOCATION: See Figure 2

SURFACE ELEVATION: 115.20 ± feet
CASING ELEVATION ± feet

DATE STARTED: 4/3/2020
DATE COMPLETED: 4/3/2020
LOGGED BY: C.Parks

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION
0		SM	Loose, Dark olive brown, gravelly, silty SAND, moist. **(COLLUVIUM)**
		ML	Medium dense, olive gray with clumps of rust mottling, sandy SILT, moist. Less rust mottling with up to 2" gravel starting at 26"
		ML	Dense to very dense, olive gray, clayey SILT, moist. Light groundwater seepage at 6 feet. **(TRANSITIONAL BEDS)**
5			
		CL	Very stiff to hard, gray to dark gray, silty CLAY, moist.

Handhole and DCP terminated at 7 feet.
Groundwater seepage observed at 6 feet during excavation.

▲ Non-Standard Penetration Resistance (Blows per foot)
(17.6 lb. weight, 22.6" drop)

SAMPLE TYPE
SAMPLE NUMBER
PEN. RESISTANCE (blows/6 inches)
OTHER TESTS
GROUNDWATER

PEN. RESISTANCE (blows/6 inches): 0 10 20 30 40 50

Water Content (%): 0 20 40 60 80 100

Plastic Limit ├──●──┤ Liquid Limit
Natural Water Content

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GeoSciences Inc.

Edgewater Creek Bridge
Everett, Washington

HAND HOLE:
HH-3
PAGE: 1 of 1

PROJECT NO.: 2019-157-21 FIGURE: A-33

HAND HOLE WITH DCP TO SPT 2019-157-21.GPJ 6/24/20

DRILLING COMPANY: HWA GeoSciences Inc.
DRILLING METHOD: Hand Bucket Auger
SAMPLING METHOD: Grab
LOCATION: See Figure 2

SURFACE ELEVATION: 87.70 ± feet
CASING ELEVATION ± feet

DATE STARTED: 3/16/2020
DATE COMPLETED: 3/16/2020
LOGGED BY: C.Parks

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION
0		ML	Loose to medium dense, olive gray, fine sandy SILT, wet. **(COLLUVIUM)**
		ML	Stiff, gray, clayey SILT, moist. Clumps of clay with rust mottling at 40-48". Grades from moist to wet at 66" with 3" of standing water at bottom of hand hole.
5			
10			

Handhole and DCP terminated at 7 feet.
Perched groundwater seepage observed in upper 1 foot.

▲ Non-Standard Penetration Resistance (Blows per foot)
(17.6 lb. weight, 22.6" drop)

PEN. RESISTANCE (blows/6 inches)

SAMPLE TYPE
SAMPLE NUMBER
OTHER TESTS
GROUNDWATER

Water Content (%)

Plastic Limit ⊢●⊣ Liquid Limit
Natural Water Content

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated
and therefore may not necessarily be indicative of other times and/or locations.

HWA GeoSciences Inc.

Edgewater Creek Bridge
Everett, Washington

HAND HOLE:
HH-4

PAGE: 1 of 1

PROJECT NO.: 2019-157-21   FIGURE: **A-34**

APPENDIX B

LABORATORY PROGRAM

APPENDIX B

LABORATORY PROGRAM

Representative soil samples obtained from our explorations were placed in plastic bags to prevent loss of moisture and transported to our Bothell, Washington, laboratory for further examination and testing. Laboratory tests were conducted on selected soil samples to characterize relevant engineering and index properties of the site soils. Laboratory testing was conducted as described below: A Summary of Material Properties is provided on Figures B-1 and B-6.

MOISTURE CONTENT, ASH, AND ORGANIC MATTER: Selected samples were tested in general accordance with method ASTM D 2974, using moisture content method 'A' (oven dried at 105^0 C) and ash content method 'C' (burned at 440^0 C). The test results are presented on the attached Summary of Material Properties, Figures B-1 through B-6. The results are percent by weight of dry soil.

PARTICLE SIZE ANALYSIS OF SOILS: Selected samples were tested to determine the particle (grain) size distribution of material in general accordance with ASTM D 422. The results are summarized on the attached Particle Size Analysis of Soils report, Figures B-7 through B-18, which also provide information regarding the classification of the sample, and the moisture content at the time of testing.

LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX OF SOILS (ATTERBERG LIMITS): Selected samples were tested using method ASTM D 4318, multi-point method. The results are reported on the attached Liquid Limit, Plastic Limit, and Plasticity Index report, Figures B-19 through B-21.

ONE DIMENSIONAL CONSOLIDATION PROPERTIES OF SOIL: The consolidation properties of BH-2A, S-9B and BH-3A, S-9B was measured in general accordance with ASTM D 2435. Saturation was maintained by inundation of the sample throughout the test. The sample was subjected to increasing increments of total stress. Each stress increment was maintained for a period of 24-hours to collect sufficient data for use in the estimation of secondary consolidation. Unloading of the sample was carried out incrementally. The test results are presented on the attached Consolidation Test Report, Figures B-22 and B-23.

EXPLORATION DESIGNATION	TOP DEPTH (feet)	BOTTOM DEPTH (feet)	MOISTURE CONTENT (%)	ORGANIC CONTENT (%)	SPECIFIC GRAVITY	ATTERBERG LIMITS (%)			% GRAVEL	% SAND	% FINES	ASTM SOIL CLASSIFICATION	SAMPLE DESCRIPTION
						LL	PL	PI					
BH-01,S-3a	7.5	8.0	15.2						11.7	74.2	14.2	SM	Olive-brown, silty SAND
BH-01,S-3b	8.0	9.0	12.4						53.7	37.0	9.2	GP-GM	Olive-brown, poorly graded GRAVEL with silt and sand
BH-01,S-6	15.0	16.5	17.3									ML	Gray, SILT with sand
BH-01,S-8	20.0	21.0	13.1									CL	Gray, lean CLAY with sand
BH-01,S-9	25.0	26.5	31.8									CL	Dark gray, lean CLAY
BH-01,S-10	30.0	31.5	29.2			39	25	14			91.5	ML	Gray, SILT
BH-01,S-11	35.0	36.5	31.1									CL	Gray, lean CLAY
BH-01,S-12	40.0	41.5	28.6							25.3	74.7	ML	Olive-brown, SILT with sand
BH-01,S-13	45.0	46.0	25.6									ML	Grayish-brown, SILT with sand
BH-01,S-14	50.0	51.0	26.2									CL	Gray, lean CLAY
BH-01,S-15	55.0	56.5	27.2						0.1	28.4	71.5	ML	Dark gray, SILT with sand
BH-01,S-16	60.0	61.5	33.3									CL	Gray, lean CLAY
BH-01,S-17	65.0	66.0	29.4			40	24	16			91.0	CL	Gray, lean CLAY
BH-01,S-18	70.0	71.0	30.6									CL	Gray, lean CLAY
BH-01,S-19	75.0	76.0	29.6									CL	Gray, lean CLAY
BH-01,S-20	80.0	81.5	25.0									SM	Dark gray, silty SAND
BH-01,S-22	90.0	91.5	26.1									SM	Dark gray, silty SAND
BH-01,S-23	95.0	95.9	18.8						1.2	71.1	27.7	SM	Very dark gray, silty SAND
BH-02a,S-1	0.0	1.5	32.1									ML	Gray, SILT with organics
BH-02a,S-2a	10.0	10.5	29.9									ML	Grayish-brown, SILT

Notes: 1. This table summarizes information presented elsewhere in the report and should be used in conjunction with the report test, other graphs and tables, and the exploration logs.

2. The soil classifications in this table are based on ASTM D2487 and D2488 as applicable.

HWA GeoSciences Inc.

Edgewater Creek Bridge
Everett, Washington

SUMMARY OF MATERIAL PROPERTIES

PAGE: 1 of 6

PROJECT NO.: 2019-157-21   FIGURE: B-1

EXPLORATION DESIGNATION	TOP DEPTH (feet)	BOTTOM DEPTH (feet)	MOISTURE CONTENT (%)	ORGANIC CONTENT (%)	SPECIFIC GRAVITY	ATTERBERG LIMITS (%)			% GRAVEL	% SAND	% FINES	ASTM SOIL CLASSIFICATION	SAMPLE DESCRIPTION
						LL	PL	PI					
BH-02a,S-4	15.0	16.5	29.0			50	28	22			66.0	CH	Olive-brown, sandy fat CLAY
BH-02a,S-6	20.0	21.5	29.2									ML	Gray, SILT
BH-02a,S-8	25.0	26.5	26.2							34.0	66.0	ML	Gray, sandy SILT
BH-02a,S-9b	30.9	31.4	25.0			25	20	5			44.7	SC	Dark gray, clayey SAND
BH-02a,S-11	35.0	36.5	15.7						4.0	85.5	10.5	SW-SM	Dark gray, well-graded SAND with silt
BH-02a,S-12	40.0	41.0	27.3									SM	Gray, silty SAND
BH-02a,S-13	45.0	46.0	20.7									SM	Gray, silty SAND
BH-02a,S-15	50.0	50.5	19.3									SM	Gray, silty SAND
BH-02a,S-17	55.0	56.5	21.5						0.1	79.6	20.4	SM	Dark gray, silty SAND
BH-02a,S-21	65.0	66.5	22.1									SM	Gray, silty SAND
BH-02a,S-24	70.0	71.5	28.4			39	29	10			64.1	ML	Grayish-brown, sandy SILT
BH-02a,S-27	75.0	75.9	13.8									ML	Dark gray, SILT with sand
BH-03a,S-2	3.0	4.0	28.6									ML	Light olive-brown, SILT
BH-03a,S-3b	5.5	6.0	30.4			38	23	15			71.5	CL	Olive-brown, lean CLAY with sand
BH-03a,S-4	7.5	8.0	28.7									ML	Gray, SILT
BH-03a,S-5	10.0	11.5	33.4									ML	Light olive-brown, SILT
BH-03a,S-7	15.0	16.5	27.8			26	21	5			68.8	CL-ML	Olive-brown, sandy silty CLAY
BH-03a,S-9a	20.0	21.0	29.0							45.1	54.9	ML	Grayish-brown, sandy SILT
BH-03a,S-11	25.0	26.5	24.6									SM	Dark gray, silty SAND
BH-03a,S-13	30.0	31.5	23.7									SM	Dark gray, silty SAND

Notes: 1. This table summarizes information presented elsewhere in the report and should be used in conjunction with the report test, other graphs and tables, and the exploration logs.

2. The soil classifications in this table are based on ASTM D2487 and D2488 as applicable.

HWA GeoSciences Inc.

INDEX MATSUM 2 2019-157-21.GPJ 7/30/21

Edgewater Creek Bridge
Everett, Washington

SUMMARY OF
MATERIAL PROPERTIES

PAGE: 2 of 6

PROJECT NO.: 2019-157-21   FIGURE: B-2

EXPLORATION DESIGNATION	TOP DEPTH (feet)	BOTTOM DEPTH (feet)	MOISTURE CONTENT (%)	ORGANIC CONTENT (%)	SPECIFIC GRAVITY	ATTERBERG LIMITS (%)			% GRAVEL	% SAND	% FINES	ASTM SOIL CLASSIFICATION	SAMPLE DESCRIPTION
						LL	PL	PI					
BH-03a,S-16	35.0	36.5	22.7									SM	Dark gray, silty SAND
BH-03a,S-18	40.0	41.5	26.5						0.6	78.9	20.5	SM	Dark gray, silty SAND
BH-03a,S-22	50.0	51.5	23.6									SM	Gray, silty SAND
BH-03a,S-23	53.0	54.0	26.0						5.0	80.6	14.3	SM	Dark gray, silty SAND
BH-03a,S-25a	60.0	60.5	20.6									SM	Dark gray, silty SAND
BH-03a,S-27	65.0	65.9	12.8						23.2	65.8	11.0	SW-SM	Grayish-brown, well-graded SAND with silt and gravel
BH-03a,S-30	70.0	71.5	24.7			29	23	6			39.6	SM	Dark grayish-brown, silty SAND
BH-03a,S-31	72.5	73.0	26.8									ML	Dark gray, SILT
BH-04,S-1	0.3	1.8	4.3						26.4	57.4	16.1	SM	Dark yellowish-brown, silty SAND with gravel
BH-04,S-5	10.0	11.5	42.9									CL	Light olive-brown, lean CLAY
BH-04,S-9	20.0	21.5	36.0			52	28	24			98.7	CH	Gray, fat CLAY
BH-04,S-10	25.0	26.5	30.3									CL	Gray, lean CLAY
BH-04,S-12	35.0	36.5	27.6						0.9	30.0	69.1	ML	Grayish-brown, sandy SILT
BH-04,S-13	40.0	41.0	23.4									ML	Grayish-brown, SILT with sand
BH-04,S-14	45.0	46.0	25.6							20.3	79.7	ML	Grayish-brown, SILT with sand
BH-04,S-15	50.0	51.5	27.2									CL	Grayish-brown, lean CLAY
BH-04,S-17	60.0	61.4	26.9			40	25	15			87.9	CL	Gray, lean CLAY
BH-04,S-20	75.0	76.5	28.3									CL	Grayish-brown, lean CLAY with sand
BH-04,S-21	80.0	81.5	27.7									SM	Dark gray, silty SAND
BH-04,S-23	90.0	91.5	25.3									SM	Dark gray, silty SAND

Notes: 1. This table summarizes information presented elsewhere in the report and should be used in conjunction with the report test, other graphs and tables, and the exploration logs.

2. The soil classifications in this table are based on ASTM D2487 and D2488 as applicable.

HWA GeoSciences Inc.

Edgewater Creek Bridge
Everett, Washington

SUMMARY OF
MATERIAL PROPERTIES

PAGE: 3 of 6

PROJECT NO.: 2019-157-21   FIGURE: B-3

EXPLORATION DESIGNATION	TOP DEPTH (feet)	BOTTOM DEPTH (feet)	MOISTURE CONTENT (%)	ORGANIC CONTENT (%)	SPECIFIC GRAVITY	ATTERBERG LIMITS (%)			% GRAVEL	% SAND	% FINES	ASTM SOIL CLASSIFICATION	SAMPLE DESCRIPTION
						LL	PL	PI					
BH-05,S-1	1.0	2.0	6.3									SM	Olive-brown, silty SAND with gravel
BH-05,S-2	5.0	6.5	38.1			56	31	25			92.7	MH	Light olive-brown, elastic SILT
BH-05,S-5	10.0	11.5	29.4									ML	Gray, SILT
BH-05,S-8	15.0	16.5	32.2									ML	Gray, SILT
BH-05,S-11	20.0	21.5	27.7									ML	Gray, SILT
BH-05,S-13	25.0	26.5	27.4									ML	Gray, SILT
BH-05,S-15	30.0	31.5	27.2									ML	Gray, SILT
BH-06,S-1	2.5	4.0	20.2									ML	Light olive-brown, SILT with sand
BH-06,S-2	5.0	6.5	33.5									ML	Light olive-brown, SILT
BH-06,S-3	7.5	9.0	31.6			45	28	17			99.0	ML	Gray, SILT
BH-06,S-5	12.5	14.0	32.7									ML	Gray, SILT
BH-06,S-6a	14.0	14.8	27.9									ML	Gray, SILT
BH-07,S-1a	2.5	3.5	10.3									ML	Olive-brown, sandy SILT with gravel
BH-07,S-2a	5.0	5.3	30.3									ML	Olive-brown, SILT
BH-07,S-3	7.5	9.0	34.4			47	29	18			90.0	ML	Olive-brown, SILT
BH-07,S-4	10.0	11.5	27.8									ML	Gray, SILT
BH-07,S-5	12.5	14.0	27.8									ML	Gray, SILT
BH-08,S-1	2.5	4.0	24.0									ML	Grayish-brown, sandy SILT
BH-08,S-3A	7.5	8.5	23.2									ML	Grayish-brown, sandy SILT
BH-08,S-6	13.0	15.0	18.2						8.3	75.5	16.3	SM	Dark gray, silty SAND

Notes: 1. This table summarizes information presented elsewhere in the report and should be used in conjunction with the report test, other graphs and tables, and the exploration logs.

2. The soil classifications in this table are based on ASTM D2487 and D2488 as applicable.

HWA GeoSciences Inc.

Edgewater Creek Bridge
Everett, Washington

SUMMARY OF
MATERIAL PROPERTIES

PAGE: 4 of 6

PROJECT NO.: 2019-157-21   FIGURE: B-4

EXPLORATION DESIGNATION	TOP DEPTH (feet)	BOTTOM DEPTH (feet)	MOISTURE CONTENT (%)	ORGANIC CONTENT (%)	SPECIFIC GRAVITY	ATTERBERG LIMITS (%)			% GRAVEL	% SAND	% FINES	ASTM SOIL CLASSIFICATION	SAMPLE DESCRIPTION
						LL	PL	PI					
BH-09,S-1	2.5	4.0	36.9			40	27	13				ML	Grayish-brown, SILT
BH-09,S-3	7.5	9.0	37.7									ML	Dark gray, SILT
BH-09,S-5	12.5	14.0	24.2									SM	Dark gray, silty SAND
BH-10,S-1	2.5	4.0	9.6									SM	Dark yellowish-brown, silty SAND
BH-10,S-2	5.0	6.5	31.8			45	30	15				ML	Grayish-brown, SILT
BH-10,S-3	7.5	9.0	27.7									ML	Dark gray, SILT
BH-11,S-1	2.5	4.0	23.0							2.4	97.6	ML	Light brownish-gray, SILT
BH-11,S-2	5.0	6.5	35.4									CL	Light olive-brown, lean CLAY
BH-11,S-3	7.5	9.0	26.4									ML	Light brownish-gray, SILT
BH-12,S-2	5.0	6.5	10.6						33.8	52.0	14.2	SM	Olive-brown, silty SAND with gravel
BH-12,S-3	7.5	9.0	37.4									MH	Light olive-brown, elastic SILT
BH-12,S-4	10.0	11.5	36.9									MH	Olive-brown, elastic SILT
BH-12,S-5	12.5	14.0	34.2									MH	Gray, elastic SILT
BH-12,S-6	15.0	16.5	33.7			56	32	24			98.0	MH	Gray, elastic SILT
BH-12,S-8	25.0	26.5	29.4									CL	Gray, lean CLAY
BH-12,S-10	35.0	36.5	28.3									CL	Grayish-brown, lean CLAY
BH-12,S-11	40.0	41.5	29.8									CL	Grayish-brown, lean CLAY
BH-13,S-3b	8.0	9.0	29.7									CL	Light olive-brown, lean CLAY
BH-13,S-5	12.5	14.0	33.1									MH	Gray, elastic SILT
BH-13,S-6	15.0	16.5	33.7									MH	Gray, elastic SILT

Notes: 1. This table summarizes information presented elsewhere in the report and should be used in conjunction with the report test, other graphs and tables, and the exploration logs.

2. The soil classifications in this table are based on ASTM D2487 and D2488 as applicable.

HWA GeoSciences Inc.

Edgewater Creek Bridge
Everett, Washington

SUMMARY OF
MATERIAL PROPERTIES

PROJECT NO.: 2019-157-21   FIGURE: B-5

INDEX MATSUM 2 2019-157-21.GPJ 7/30/21

EXPLORATION DESIGNATION	TOP DEPTH (feet)	BOTTOM DEPTH (feet)	MOISTURE CONTENT (%)	ORGANIC CONTENT (%)	SPECIFIC GRAVITY	ATTERBERG LIMITS (%)			% GRAVEL	% SAND	% FINES	ASTM SOIL CLASSIFICATION	SAMPLE DESCRIPTION
						LL	PL	PI					
BH-13,S-7	20.0	21.5	34.9									MH	Gray, elastic SILT
BH-13,S-9	30.0	31.5	26.8			40	25	15			99.8	CL	Gray, lean CLAY
BH-13,S-10	35.0	36.5	24.7									ML	Light brownish-gray, SILT with sand
BH-13,S-11	40.0	41.5	27.6									SM	Light brownish-gray, silty SAND
BH-13,S-12	45.0	46.5	7.4							65.7	34.3	SM	Light brownish-gray, silty SAND

Notes: 1. This table summarizes information presented elsewhere in the report and should be used in conjunction with the report test, other graphs and tables, and the exploration logs.

2. The soil classifications in this table are based on ASTM D2487 and D2488 as applicable.

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Edgewater Creek Bridge
Everett, Washington

SUMMARY OF
MATERIAL PROPERTIES

PAGE: 6 of 6

PROJECT NO.: 2019-157-21   FIGURE: B-6
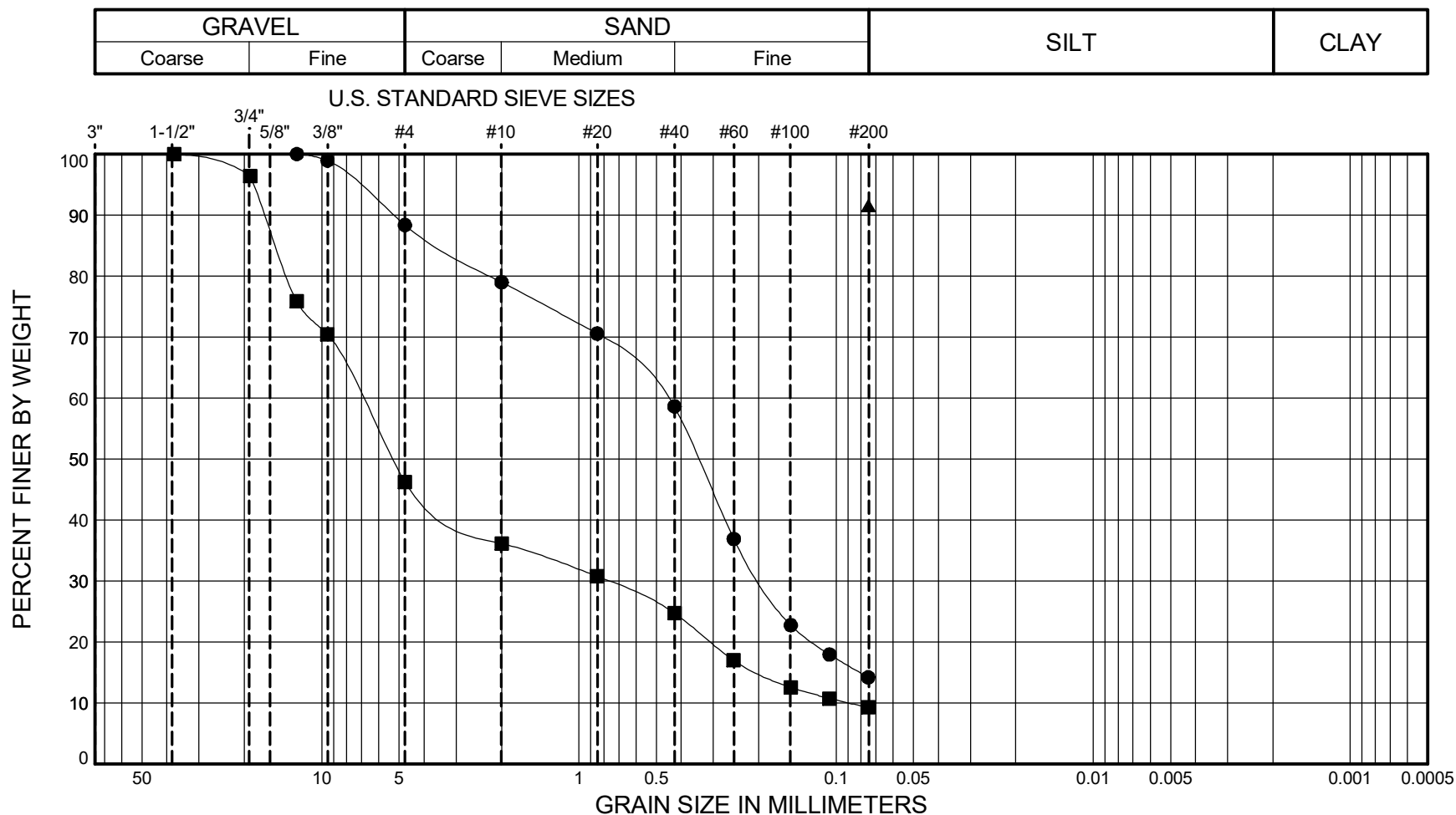
INDEX MATSUM 2 2019-157-21.GPJ 7/30/21

			GRAVEL		SAND			SILT		CLAY

GRAVEL: Coarse | Fine
SAND: Coarse | Medium | Fine
SILT
CLAY

U.S. STANDARD SIEVE SIZES

3" 1-1/2" 3/4" 5/8" 3/8" #4 #10 #20 #40 #60 #100 #200

PERCENT FINER BY WEIGHT

GRAIN SIZE IN MILLIMETERS

SYMBOL	SAMPLE		CLASSIFICATION OF SOIL- ASTM D2487 Group Symbol and Name	% MC	D85	D60	D30	D15	D10	Gravel %	Sand %	Silt %	Clay %
●	BH-01	S-3a	(SM) Olive-brown, silty SAND	15	3.49	0.46	0.19	0.08		11.7	74.2	14.2	
■	BH-01	S-3b	(GP-GM) Olive-brown, poorly graded GRAVEL with silt and sand	12	15.06	7.04	0.78	0.20	0.0894	53.7	37.0	9.2	
▲	BH-01	S-10	(ML) Gray, SILT	29								91.5	

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Edgewater Creek Bridge
Everett, Washington

PARTICLE-SIZE ANALYSIS
OF SOILS
METHOD ASTM D422

PROJECT NO.: 2019-157-21   FIGURE: B-7

HWA GRSZ D85-D60-D30-D10-D15 2019-157-21.GPJ 7/30/21

PARTICLE-SIZE ANALYSIS
OF SOILS
METHOD ASTM D422

Edgewater Creek Bridge
Everett, Washington

SYMBOL	SAMPLE		CLASSIFICATION OF SOIL- ASTM D2487 Group Symbol and Name	% MC	D85	D60	D30	D15	D10	Gravel %	Sand %	Silt %	Clay %
●	BH-01	S-12	(ML) Olive-brown, SILT with sand	29	0.16						25.3	74.7	
■	BH-01	S-15	(ML) Dark gray, SILT with sand	27	0.18					0.1	28.4	71.5	
▲	BH-01	S-17	(CL) Gray, lean CLAY	29								91.0	

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PROJECT NO.: 2019-157-21   FIGURE: B-8

	GRAVEL		SAND			SILT	CLAY
	Coarse	Fine	Coarse	Medium	Fine		

U.S. STANDARD SIEVE SIZES

Sieve labels: 3" 1-1/2" 3/4" 5/8" 3/8" #4 #10 #20 #40 #60 #100 #200

Y-axis: PERCENT FINER BY WEIGHT (0–100)

X-axis: GRAIN SIZE IN MILLIMETERS (50 ... 0.0005)

SYMBOL	SAMPLE		CLASSIFICATION OF SOIL- ASTM D2487 Group Symbol and Name	% MC	D85	D60	D30	D15	D10	Gravel %	Sand %	Silt %	Clay %
●	BH-01	S-23	(SM) Very dark gray, silty SAND	19	1.02	0.50	0.09			1.2	71.1	27.7	
■	BH-02a	S-4	(CH) Olive-brown, sandy fat CLAY	29								66.0	
▲	BH-02a	S-8	(ML) Gray, sandy SILT	26	0.16						34.0	66.0	

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Edgewater Creek Bridge
Everett, Washington

PARTICLE-SIZE ANALYSIS
OF SOILS
METHOD ASTM D422

PROJECT NO.: 2019-157-21   FIGURE: B-9

HWA GRSZ D85-D60-D30-D10-D15 2019-157-21.GPJ 7/30/21

PARTICLE-SIZE ANALYSIS
OF SOILS
METHOD ASTM D422

Edgewater Creek Bridge
Everett, Washington

SYMBOL	SAMPLE		CLASSIFICATION OF SOIL- ASTM D2487 Group Symbol and Name	% MC	D85	D60	D30	D15	D10	Gravel %	Sand %	Silt %	Clay %
●	BH-02a	S-9b	(SC) Dark gray, clayey SAND	25								44.7	
■	BH-02a	S-11	(SW-SM) Dark gray, well-graded SAND with silt	16	0.93	0.58	0.31	0.13		4.0	85.5	10.5	
▲	BH-02a	S-17	(SM) Dark gray, silty SAND	22	0.22	0.16	0.11			0.1	79.6	20.4	

PROJECT NO.: 2019-157-21   FIGURE: B-10

HWA GRSZ D85-D60-D30-D10-D15 2019-157-21.GPJ 7/30/21

	GRAVEL		SAND			SILT	CLAY
	Coarse	Fine	Coarse	Medium	Fine		

U.S. STANDARD SIEVE SIZES

3" 1-1/2" 3/4" 5/8" 3/8" #4 #10 #20 #40 #60 #100 #200

PERCENT FINER BY WEIGHT

GRAIN SIZE IN MILLIMETERS

50   10   5   1   0.5   0.1   0.05   0.01   0.005   0.001   0.0005
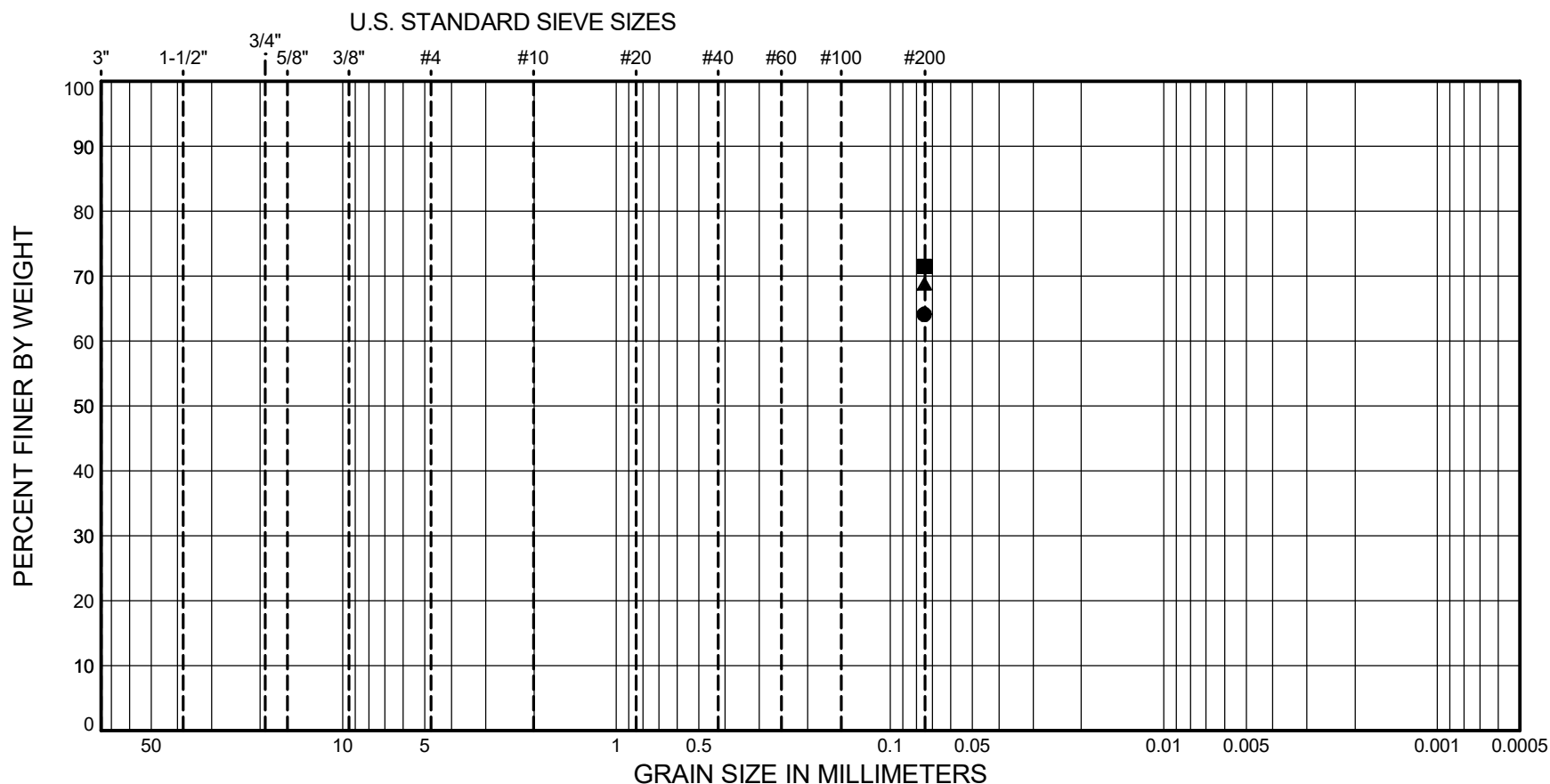
SYMBOL	SAMPLE		CLASSIFICATION OF SOIL- ASTM D2487 Group Symbol and Name	% MC	D85	D60	D30	D15	D10	Gravel %	Sand %	Silt %	Clay %
●	BH-02a	S-24	(ML) Grayish-brown, sandy SILT	28								64.1	
■	BH-03a	S-3b	(CL) Olive-brown, lean CLAY with sand	30								71.5	
▲	BH-03a	S-7	(CL-ML) Olive-brown, sandy silty CLAY	28								68.8	

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Edgewater Creek Bridge
Everett, Washington

PARTICLE-SIZE ANALYSIS
OF SOILS
METHOD ASTM D422

PROJECT NO.: 2019-157-21   FIGURE: B-11

HWA GRSZ D85-D60-D30-D10-D15 2019-157-21.GPJ 7/30/21

PARTICLE-SIZE ANALYSIS
OF SOILS
METHOD ASTM D422

Edgewater Creek Bridge
Everett, Washington

SYMBOL	SAMPLE		CLASSIFICATION OF SOIL- ASTM D2487 Group Symbol and Name	% MC	D85	D60	D30	D15	D10	Gravel %	Sand %	Silt %	Clay %
●	BH-03a	S-9a	(ML) Grayish-brown, sandy SILT	29	0.16	0.09					45.1	54.9	
■	BH-03a	S-18	(SM) Dark gray, silty SAND	27	0.23	0.16	0.11			0.6	78.9	20.5	
▲	BH-03a	S-23	(SM) Dark gray, silty SAND	26	0.25	0.19	0.13	0.08		5.0	80.6	14.3	

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PROJECT NO.: 2019-157-21   FIGURE: B-12

HWA GRSZ D85-D60-D30-D10-D15 2019-157-21.GPJ 7/30/21

		GRAVEL		SAND			SILT	CLAY
		Coarse	Fine	Coarse	Medium	Fine		

U.S. STANDARD SIEVE SIZES

3"   1-1/2"   3/4"   5/8"   3/8"   #4   #10   #20   #40   #60   #100   #200

PERCENT FINER BY WEIGHT

GRAIN SIZE IN MILLIMETERS

SYMBOL	SAMPLE		CLASSIFICATION OF SOIL- ASTM D2487 Group Symbol and Name	% MC	D85	D60	D30	D15	D10	Gravel %	Sand %	Silt %	Clay %
●	BH-03a	S-27	(SW-SM) Grayish-brown, well-graded SAND with silt and gravel	13	9.55	0.70	0.24	0.12		23.2	65.8	11.0	
■	BH-03a	S-30	(SM) Dark grayish-brown, silty SAND	25								39.6	
▲	BH-04	S-1	(SM) Dark yellowish-brown, silty SAND with gravel	4	13.16	1.45	0.23			26.4	57.4	16.1	

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Edgewater Creek Bridge
Everett, Washington

PARTICLE-SIZE ANALYSIS
OF SOILS
METHOD ASTM D422

PROJECT NO.: 2019-157-21   FIGURE: B-13
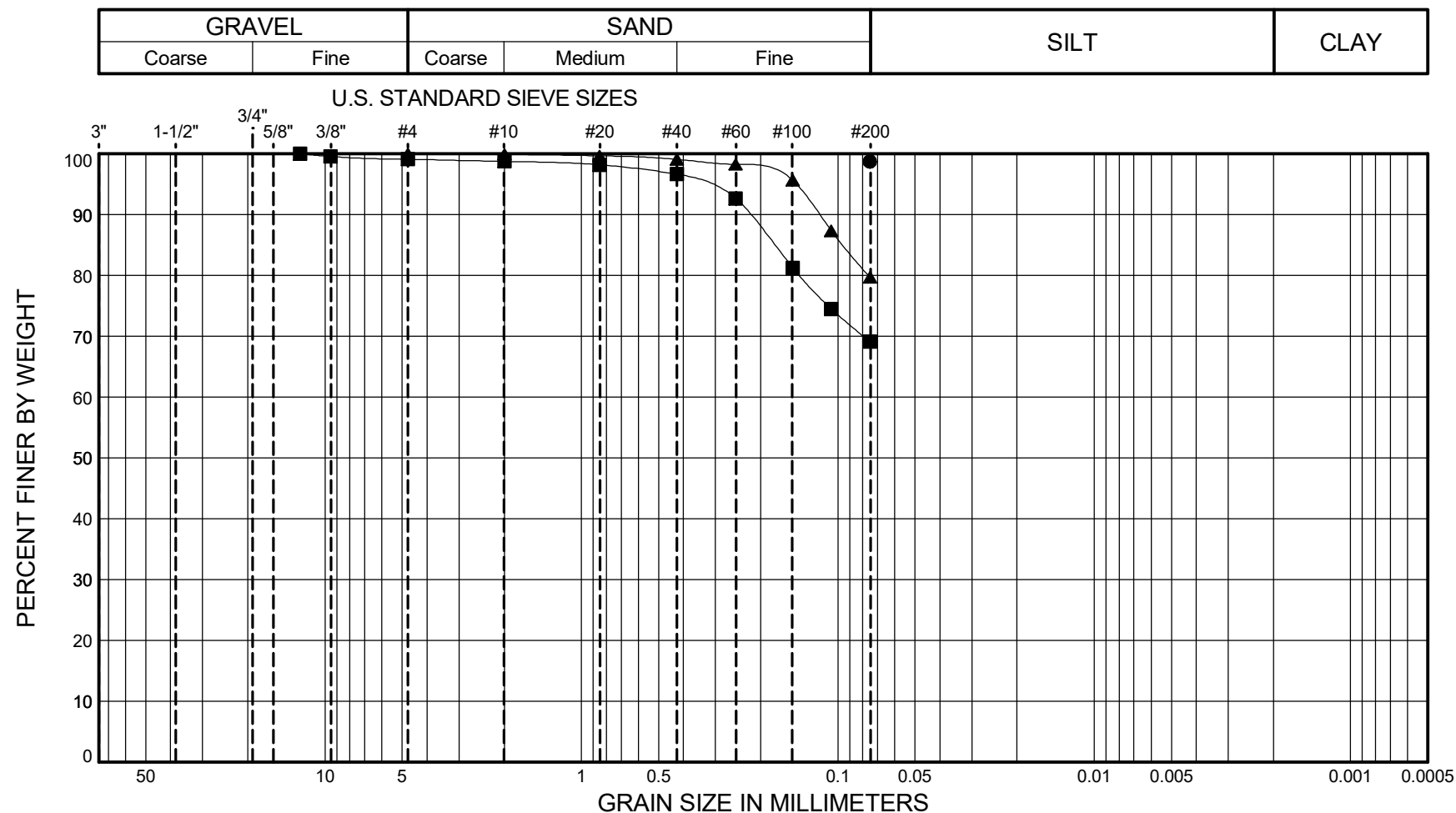
	GRAVEL		SAND			SILT	CLAY
	Coarse	Fine	Coarse	Medium	Fine		

U.S. STANDARD SIEVE SIZES

PERCENT FINER BY WEIGHT — GRAIN SIZE IN MILLIMETERS

SYMBOL	SAMPLE		CLASSIFICATION OF SOIL- ASTM D2487 Group Symbol and Name	% MC	D85	D60	D30	D15	D10	Gravel %	Sand %	Silt %	Clay %
●	BH-04	S-9	(CH) Gray, fat CLAY	36								98.7	
■	BH-04	S-12	(ML) Grayish-brown, sandy SILT	28	0.18					0.9	30.0	69.1	
▲	BH-04	S-14	(ML) Grayish-brown, SILT with sand	26	0.10						20.3	79.7	
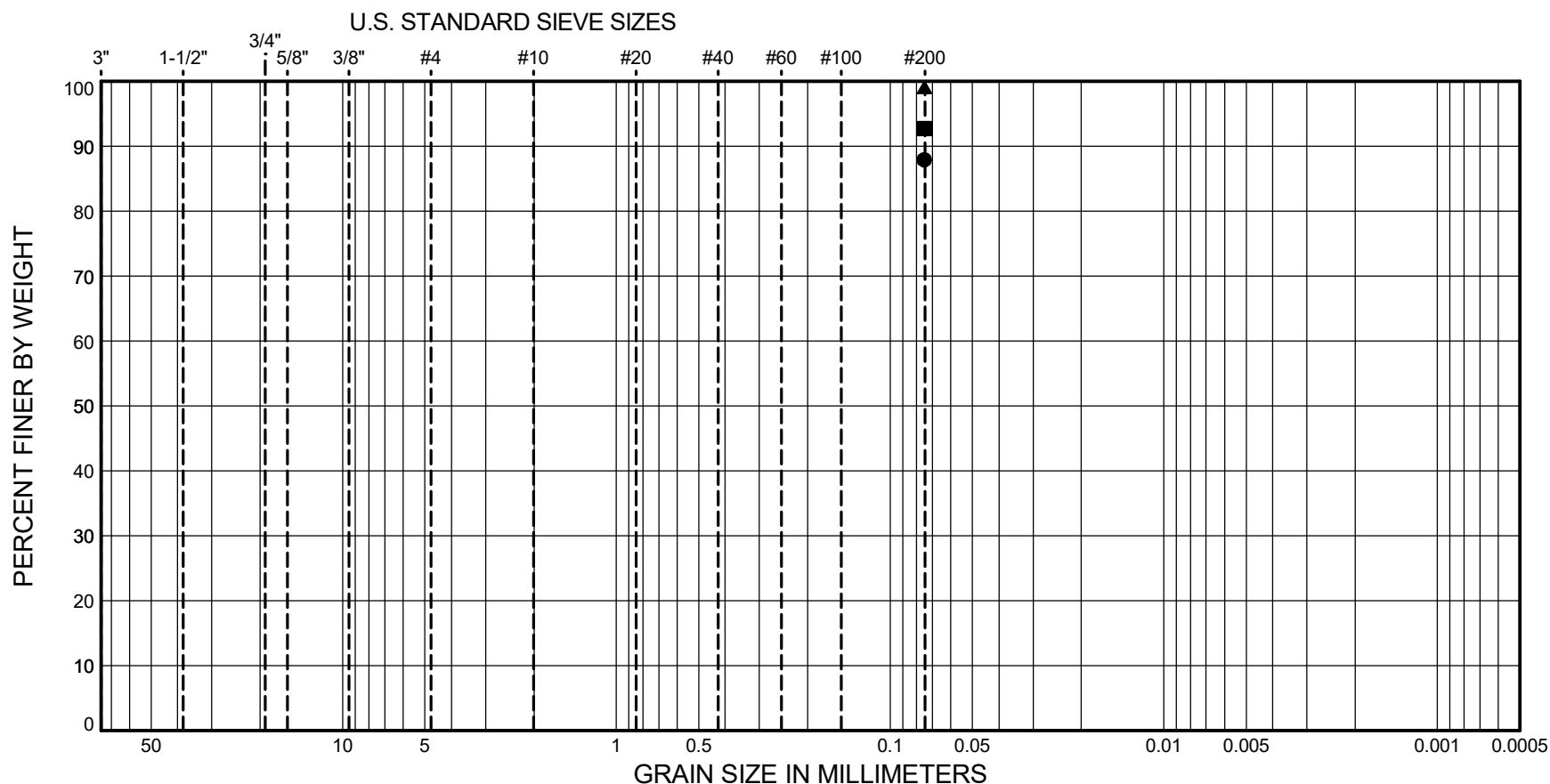
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Edgewater Creek Bridge
Everett, Washington

PARTICLE-SIZE ANALYSIS
OF SOILS
METHOD ASTM D422

PROJECT NO.: 2019-157-21   FIGURE: B-14

PARTICLE-SIZE ANALYSIS
OF SOILS
METHOD ASTM D422

Edgewater Creek Bridge
Everett, Washington

SYMBOL	SAMPLE		CLASSIFICATION OF SOIL- ASTM D2487 Group Symbol and Name	% MC	D85	D60	D30	D15	D10	Gravel %	Sand %	Silt %	Clay %
●	BH-04	S-17	(CL) Gray, lean CLAY	27								87.9	
■	BH-05	S-2	(MH) Light olive-brown, elastic SILT	38								92.7	
▲	BH-06	S-3	(ML) Gray, SILT	32								99.0	

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PARTICLE-SIZE ANALYSIS OF SOILS
METHOD ASTM D422

Edgewater Creek Bridge
Everett, Washington
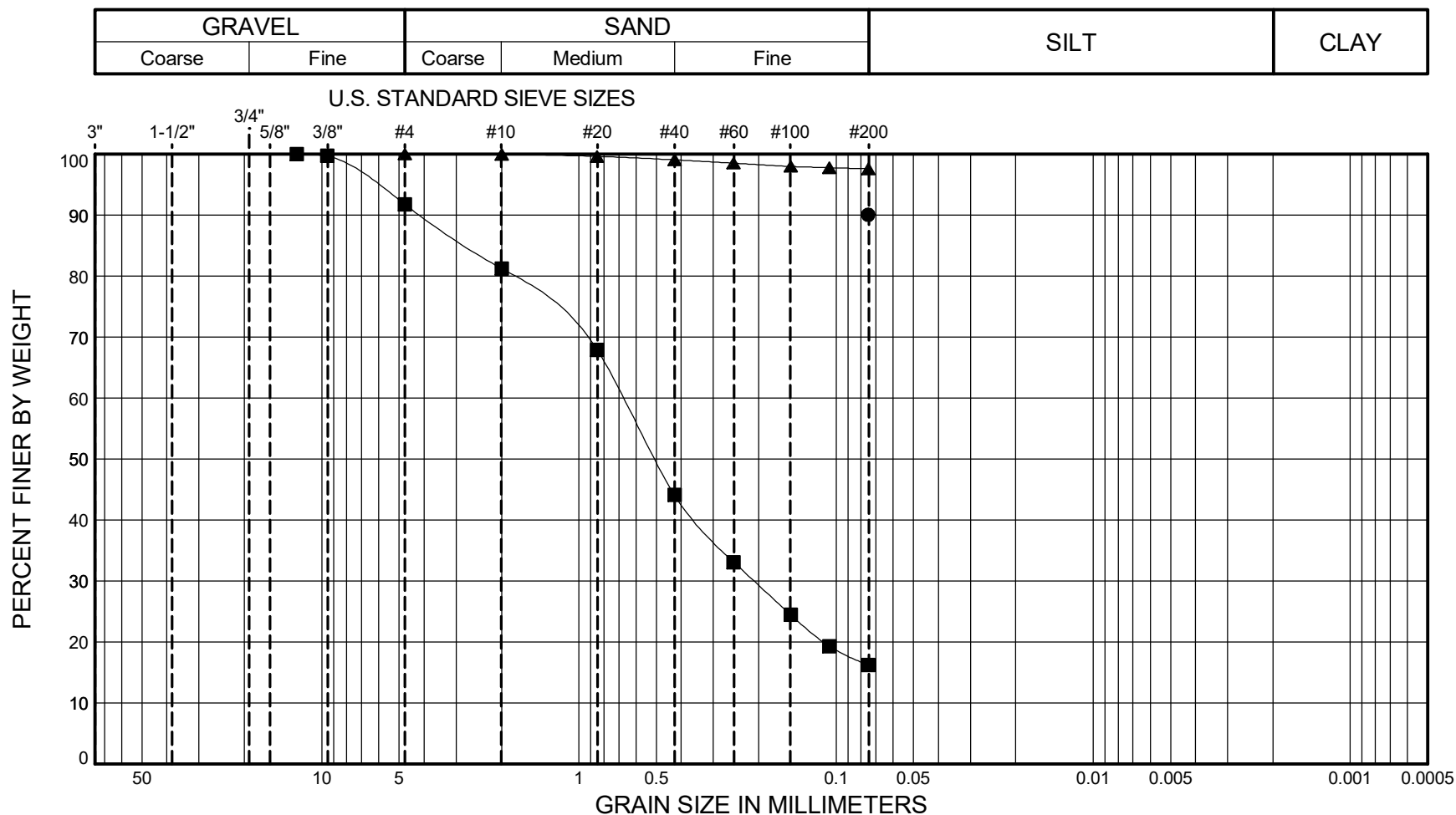
SYMBOL	SAMPLE		CLASSIFICATION OF SOIL- ASTM D2487 Group Symbol and Name	% MC	D85	D60	D30	D15	D10	Gravel %	Sand %	Silt %	Clay %
●	BH-07	S-3	(ML) Olive-brown, SILT	34								90.0	
■	BH-08	S-6	(SM) Dark gray, silty SAND	18	2.73	0.67	0.21			8.3	75.5	16.3	
▲	BH-11	S-1	(ML) Light brownish-gray, SILT	23							2.4	97.6	
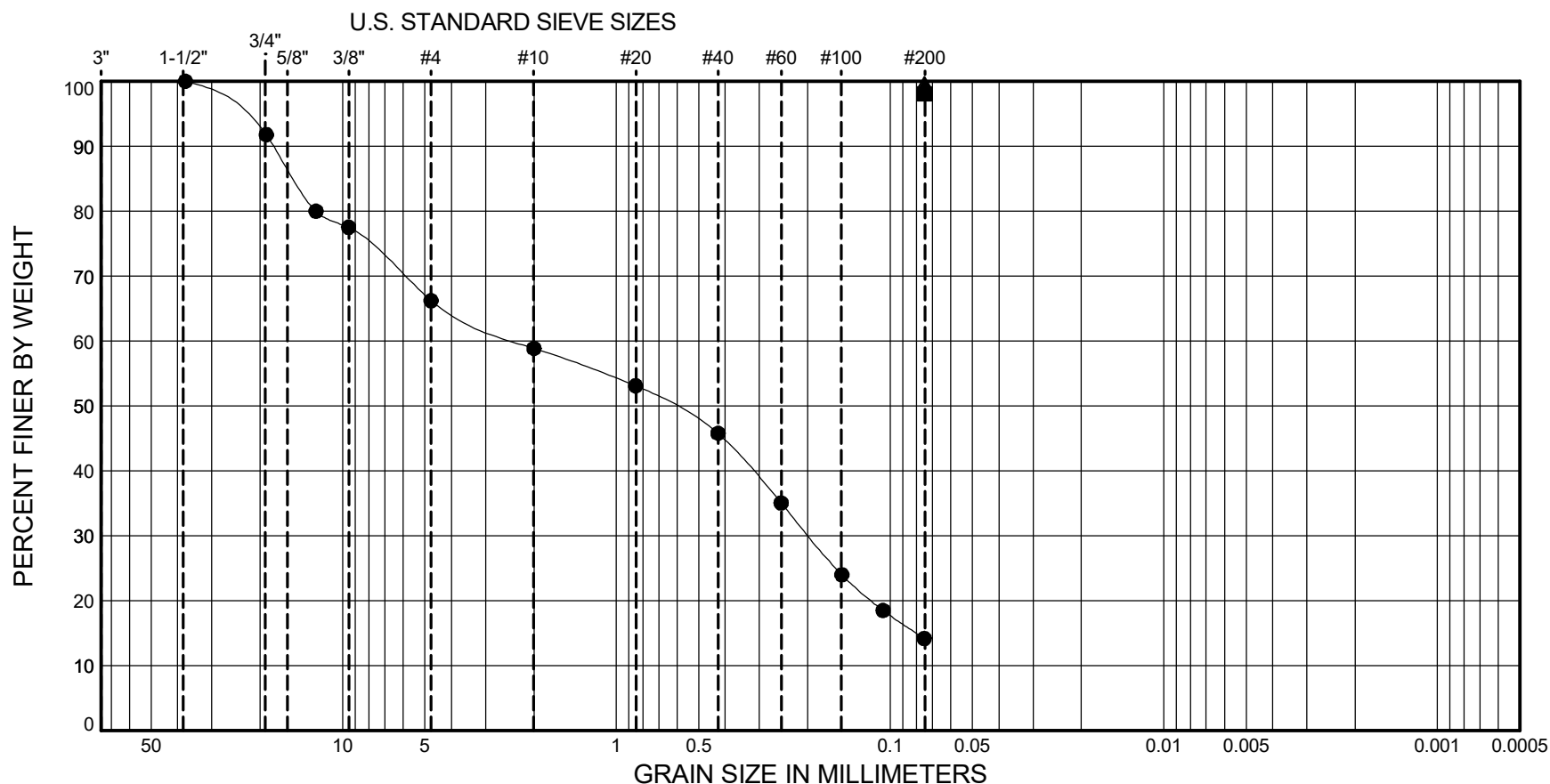
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PROJECT NO.: 2019-157-21   FIGURE: B-16

PARTICLE-SIZE ANALYSIS
OF SOILS
METHOD ASTM D422

Edgewater Creek Bridge
Everett, Washington

GRAVEL		SAND			SILT	CLAY
Coarse	Fine	Coarse	Medium	Fine		

U.S. STANDARD SIEVE SIZES

SYMBOL	SAMPLE		CLASSIFICATION OF SOIL- ASTM D2487 Group Symbol and Name	% MC	D85	D60	D30	D15	D10	Gravel %	Sand %	Silt %	Clay %
●	BH-12	S-2	(SM) Olive-brown, silty SAND with gravel	11	14.94	2.29	0.20	0.08		33.8	52.0	14.2	
■	BH-12	S-6	(MH) Gray, elastic SILT	34								98.0	
▲	BH-13	S-9	(CL) Gray, lean CLAY	27								99.8	

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PROJECT NO.: 2019-157-21   FIGURE: B-17

HWA GRSZ D85-D60-D30-D10-D15 2019-157-21.GPJ 7/30/21

	GRAVEL		SAND			SILT	CLAY
	Coarse	Fine	Coarse	Medium	Fine		

U.S. STANDARD SIEVE SIZES

GRAIN SIZE IN MILLIMETERS

PERCENT FINER BY WEIGHT
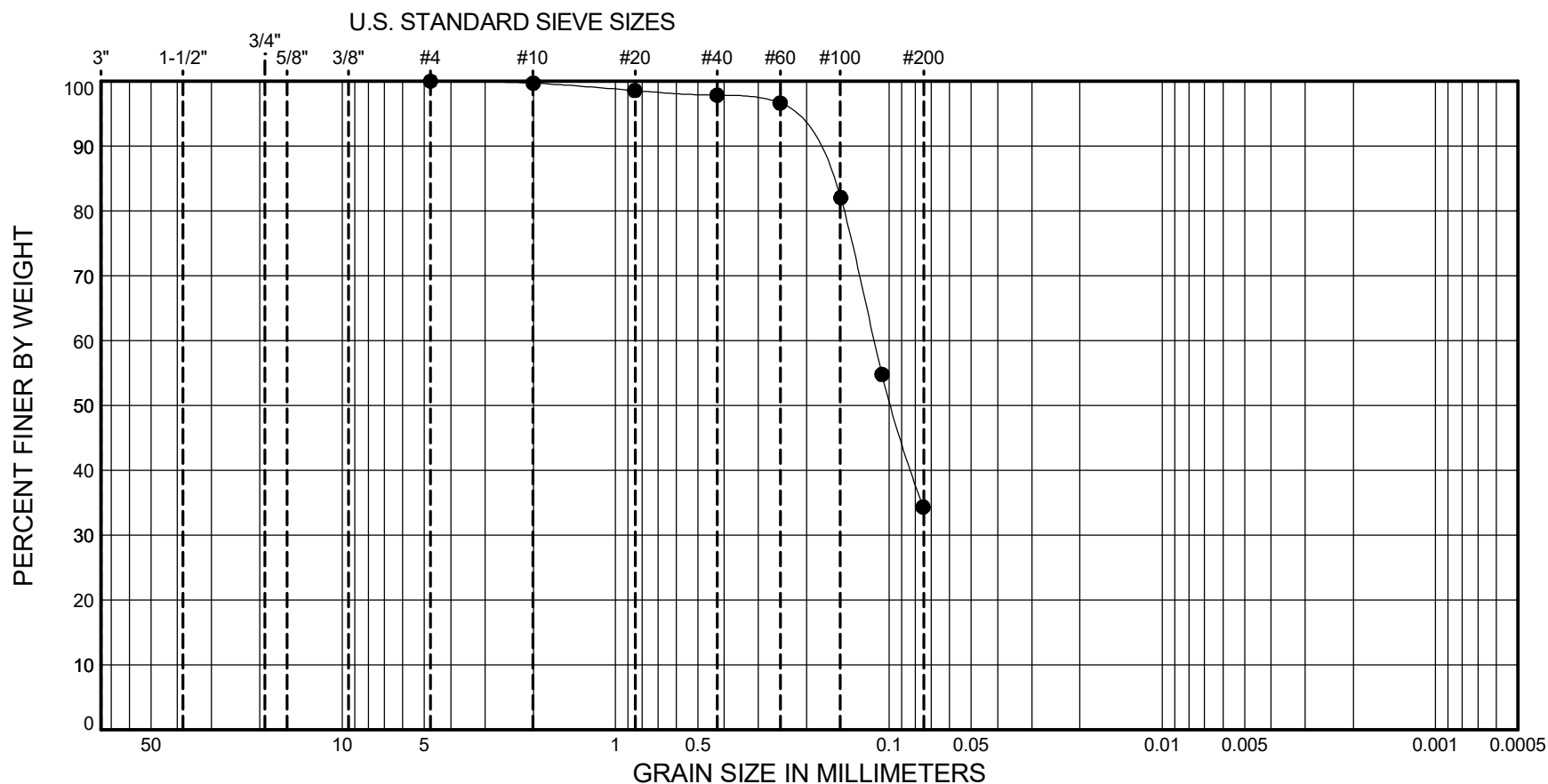
SYMBOL	SAMPLE		CLASSIFICATION OF SOIL- ASTM D2487 Group Symbol and Name	% MC	D85	D60	D30	D15	D10	Gravel %	Sand %	Silt %	Clay %
●	BH-13	S-12	(SM) Light brownish-gray, silty SAND	7	0.17	0.11					65.7	34.3	

HWA GeoSciences Inc.

Edgewater Creek Bridge
Everett, Washington

PARTICLE-SIZE ANALYSIS
OF SOILS
METHOD ASTM D422

PROJECT NO.: 2019-157-21   FIGURE: B-18

HWA GRSZ D85-D60-D30-D10-D15 2019-157-21.GPJ 7/30/21
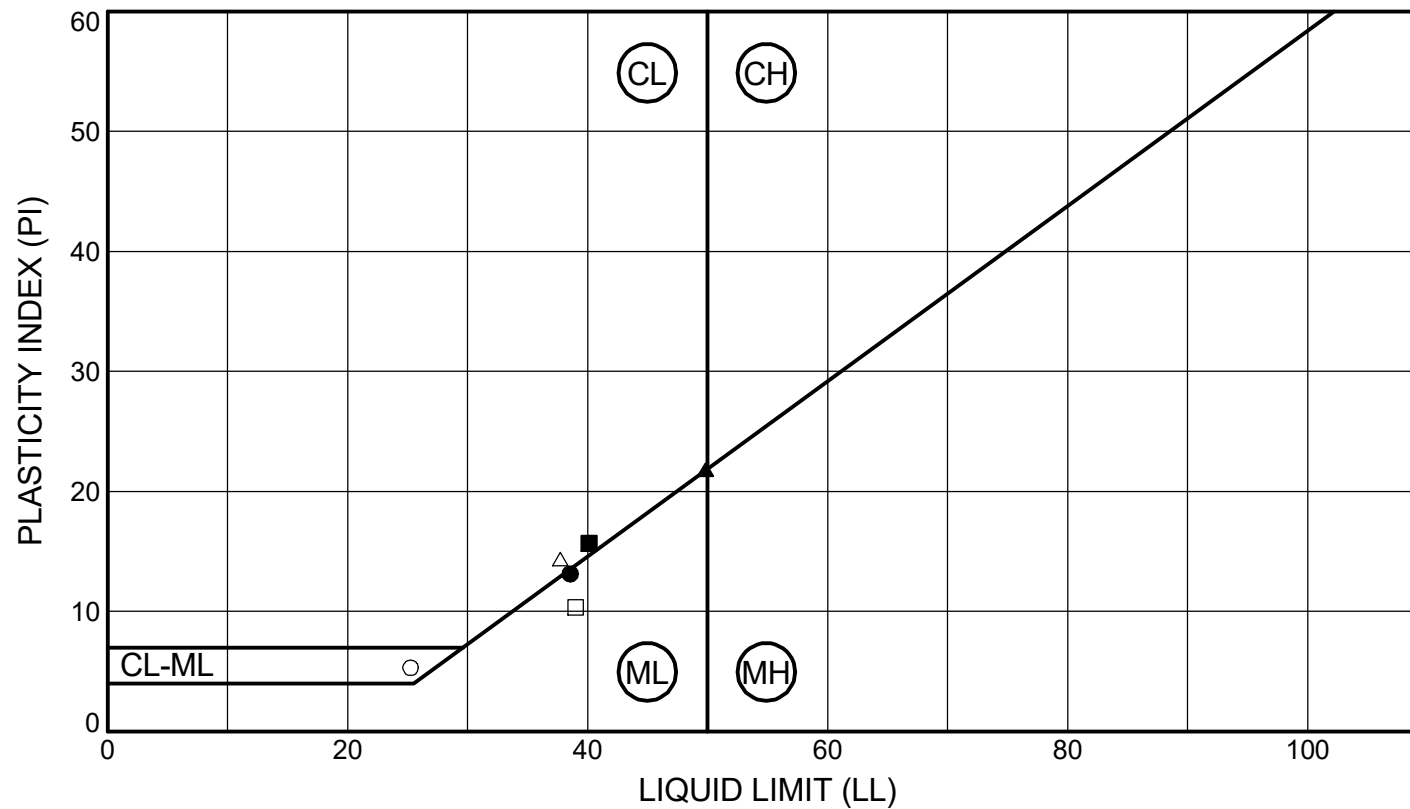
SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	LL	PL	PI	% Fines
●	BH-01	S-10	30.0 - 31.5	(ML) Gray, SILT	29	39	25	14	91.5
■	BH-01	S-17	65.0 - 66.0	(CL) Gray, lean CLAY	29	40	24	16	91.0
▲	BH-02a	S-4	15.0 - 16.5	(CH) Olive-brown, sandy fat CLAY	29	50	28	22	66.0
○	BH-02a	S-9b	30.9 - 31.4	(SC) Dark gray, clayey SAND	25	25	20	5	44.7
□	BH-02a	S-24	70.0 - 71.5	(ML) Grayish-brown, sandy SILT	28	39	29	10	64.1
△	BH-03a	S-3b	5.5 - 6.0	(CL) Olive-brown, lean CLAY with sand	30	38	23	15	71.5

HWA GeoSciences Inc.

Edgewater Creek Bridge
Everett, Washington

LIQUID LIMIT, PLASTIC LIMIT AND
PLASTICITY INDEX OF SOILS
METHOD ASTM D4318

PROJECT NO.: 2019-157-21   FIGURE: B-19

HWAATTB 2019-157-21.GPJ 7/30/21
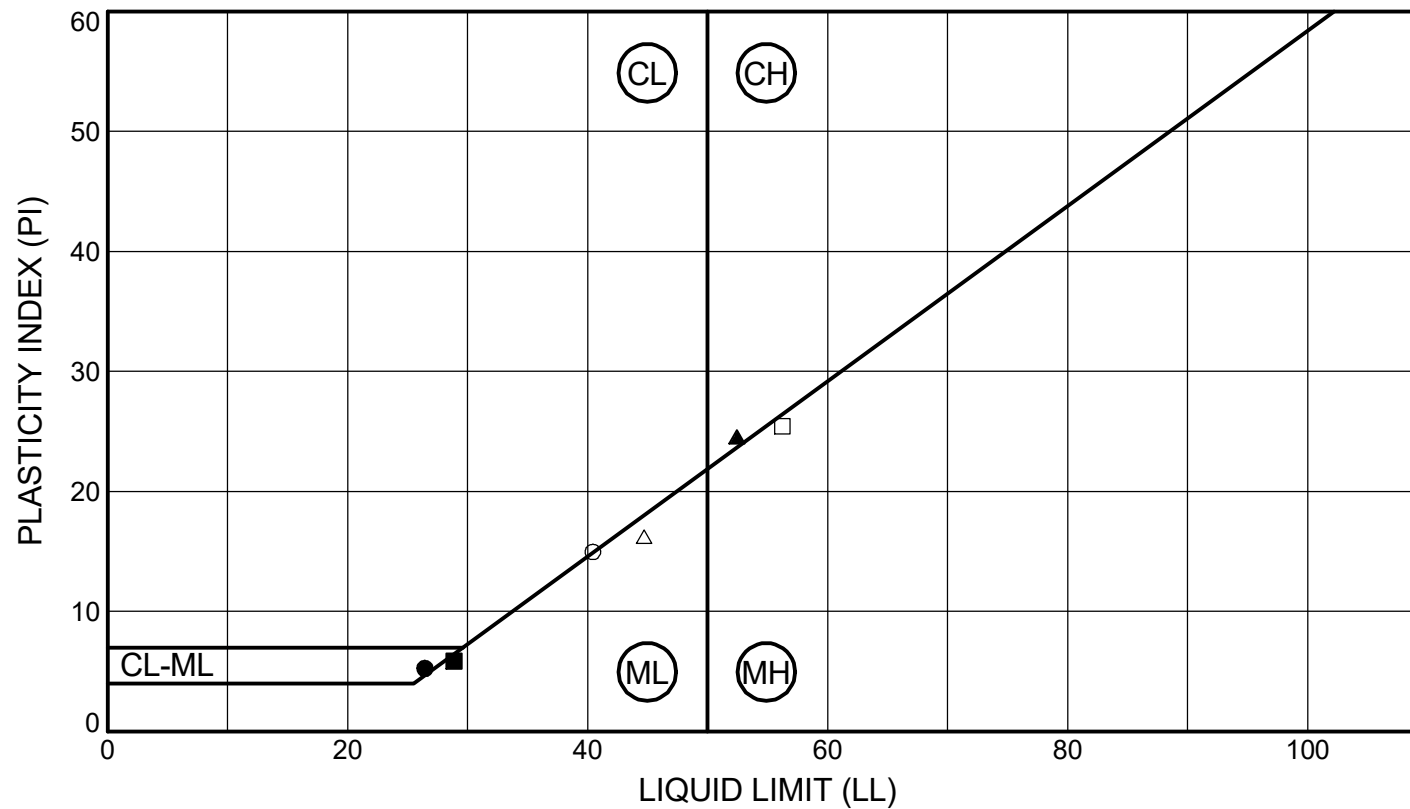
SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	LL	PL	PI	% Fines
●	BH-03a	S-7	15.0 - 16.5	(CL-ML) Olive-brown, sandy silty CLAY	28	26	21	5	68.8
■	BH-03a	S-30	70.0 - 71.5	(SM) Dark grayish-brown, silty SAND	25	29	23	6	39.6
▲	BH-04	S-9	20.0 - 21.5	(CH) Gray, fat CLAY	36	52	28	24	98.7
○	BH-04	S-17	60.0 - 61.4	(CL) Gray, lean CLAY	27	40	25	15	87.9
□	BH-05	S-2	5.0 - 6.5	(MH) Light olive-brown, elastic SILT	38	56	31	25	92.7
△	BH-06	S-3	7.5 - 9.0	(ML) Gray, SILT	32	45	28	17	99.0

HWA GeoSciences Inc.

Edgewater Creek Bridge
Everett, Washington

LIQUID LIMIT, PLASTIC LIMIT AND
PLASTICITY INDEX OF SOILS
METHOD ASTM D4318

PROJECT NO.: 2019-157-21   FIGURE: B-20

HWAATTB 2019-157-21.GPJ 7/30/21

Plasticity chart with axes PLASTICITY INDEX (PI) (0–60) vs LIQUID LIMIT (LL) (0–100), showing regions CL, CH, ML, MH, CL-ML.
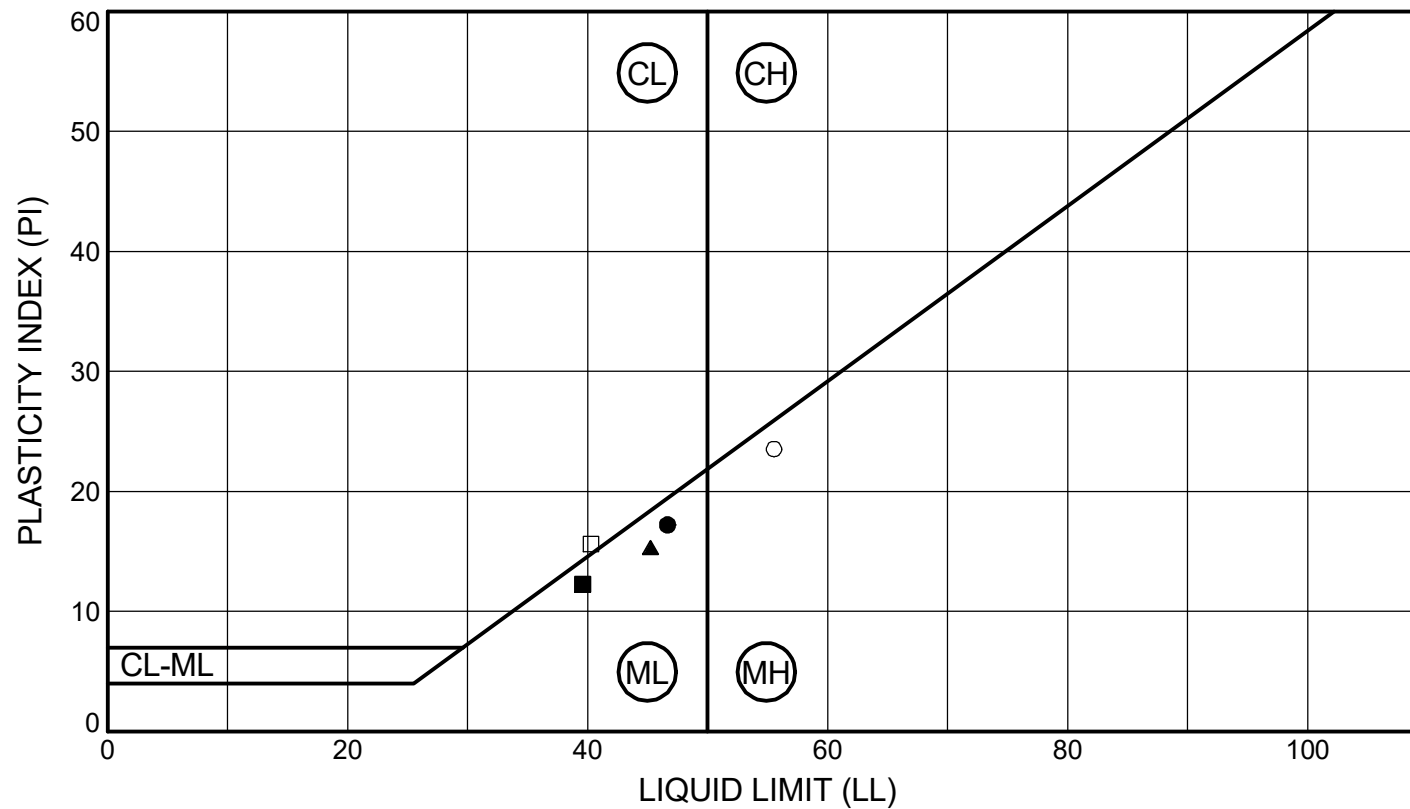
SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	LL	PL	PI	% Fines
●	BH-07	S-3	7.5 - 9.0	(ML) Olive-brown, SILT	34	47	29	18	90.0
■	BH-09	S-1	2.5 - 4.0	(ML) Grayish-brown, SILT	37	40	27	13	
▲	BH-10	S-2	5.0 - 6.5	(ML) Grayish-brown, SILT	32	45	30	15	
○	BH-12	S-6	15.0 - 16.5	(MH) Gray, elastic SILT	34	56	32	24	98.0
□	BH-13	S-9	30.0 - 31.5	(CL) Gray, lean CLAY	27	40	25	15	99.8

HWA GeoSciences Inc.

Edgewater Creek Bridge
Everett, Washington

LIQUID LIMIT, PLASTIC LIMIT AND
PLASTICITY INDEX OF SOILS
METHOD ASTM D4318

PROJECT NO.: 2019-157-21   FIGURE: B-21

HWAATTB 2019-157-21.GPJ 7/30/21

Project Name:	Edgewater Bridge			**Project Number:**	2019-157		
Sample Point:	BH-2A	**Sample No.:** S-9B		**Sample Depth:**	30.9-31.4		
Soil Description:	sandy SILT						
Soil Color:	Gray			Average Strain Rate:	0.7	% per min.	
Soil Group Symbol:	ML			Soil Specific Gravity:	2.65	(assumed)	
Normal Stress (psf)	1500.00	3000.00	6000.00	**Average**	Indicated Strength Parameters		
Peak Stress (psf)	1195.43	2269.61	4593.45				
Initial Moisture Content (%):	24.0	24.0	24.0	24.0		Cohesion	phi Angle
Wet Unit Weight (pcf):	126.2	125.6	125.7	125.8		psf	(degrees)
Dry Unit Weight (pcf):	101.7	101.3	101.4	101.5	Peak	33.5	37.2
Calculated Void Ratio	0.625	0.633	0.631	0.630			
Calculated Porosity	0.385	0.388	0.387	0.386			
Calculated Saturation (%)	101.7	100.5	100.9	101.0			
Final Moisture Content (%)	28.7	29.9	28.0	28.8			

Checked By: _____   Figure ___22___

HWA GEOSCIENCES INC. Materials Testing Laboratory

Direct Shear Test of Soils Under Consolidated Drained Conditions (ASTM D 3080)

Project Name:	Edgewater Bridge				**Project Number:**	2019-157	
Sample Point:	BH-3A	**Sample No.:** S-9B			**Sample Depth:**	21-21.5 feet	
Soil Description:	silty SAND						
Soil Color:	Gray				Average Strain Rate:	0.7	% per min.
Soil Group Symbol:	SM				Soil Specific Gravity:	2.65	(assumed)

Normal Stress (psf)	1150.00	2300.00	4600.00		**Average**	Indicated Strength Parameters	
Peak Stress (psf)	979.32	1761.51	3398.92				
Initial Moisture Content (%):	23.6	23.6	23.6		23.6	Cohesion	phi Angle
Wet Unit Weight (pcf):	125.1	126.0	123.1		124.7	psf	(degrees)
Dry Unit Weight (pcf):	101.2	101.9	99.6		100.9	Peak	160.6	35.1
Calculated Void Ratio	0.634	0.622	0.660		0.639		
Calculated Porosity	0.388	0.384	0.398		0.390		
Calculated Saturation (%)	98.6	100.4	94.7		97.9		
Final Moisture Content (%)	26.6	27.2	30.3		28.0		

Checked By: _____   Figure ___23___

APPENDIX C

CROSS SECTION C-C' (BRIDGE ALIGNMENT) GLOBAL SLOPE STABILITY ANALYSIS

Safety Factor

	0.000
	0.250
	0.500
	0.750
	1.000
	1.250
	1.500
	1.750
	2.000
	2.250
	2.500
	2.750
	3.000
	3.250
	3.500
	3.750
	4.000
	4.250
	4.500
	4.750
	5.000
	5.250
	5.500
	5.750
	6.000+

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Abutment Walls		150		
Existing Fill		120	0	34
Colluvium (Fine)		105	475	0
Transitional Beds (Fine) - Stiff		110	1500	0
Transitional Beds (Fine) - V Stiff		110	3000	0
Transitional Beds (Fine) - Hard		110	4500	0
Whidbey Formation (Coarse)		135	0	38

1.320

Project	Edgewater Creek Bridge Replacement	
Analysis Description	Figure C-1 - Cross Section A-A' - Global Slope Stability Analysis - L to R	
Drawn By SKS	**Scale** 1:700	**Company** HWA GeoSciences, Inc.
Date 5/27/2020		**Loading Scenario** Static Analysis - Existing Conditions

GEOSCIENCES INC.
DBE/MWBE
HWA

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Abutment Walls		150		
Existing Fill		120	0	34
Colluvium (Fine)		105	475	0
Transitional Beds (Fine) - Stiff		110	1500	0
Transitional Beds (Fine) - V Stiff		110	3000	0
Transitional Beds (Fine) - Hard		110	4500	0
Whidbey Formation (Coarse)		135	0	38

Safety Factor

0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+

1.331

250.00 lbs/ft2 250.00 lbs/ft2 250.00 lbs/ft2 250.00 lbs/ft2 250.00 lbs/ft2

W

Project	Edgewater Creek Bridge Replacement		
Analysis Description	Figure C-2 - Cross Section A-A' - Global Slope Stability Analysis - R to L		
Drawn By	SKS	Scale 1:600	Company HWA GeoSciences, Inc.
Date	5/27/2020	Loading Scenario	Static Analysis - Existing Conditions

HWA GeoSciences Inc.
DBE/MWBE

SLIDEINTERPRET 8.029

Figure C-3 - Cross Section A-A' - Global Slope Stability Analysis - L to R

Safety Factor	
	0.000
	0.250
	0.500
	0.750
	1.000
	1.250
	1.500
	1.750
	2.000
	2.250
	2.500
	2.750
	3.000
	3.250
	3.500
	3.750
	4.000
	4.250
	4.500
	4.750
	5.000
	5.250
	5.500
	5.750
	6.000+

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Abutment Walls		150		
Existing Fill		120	0	34
Colluvium (Fine)		105	475	0
Transitional Beds (Fine) - Stiff		110	1500	0
Transitional Beds (Fine) - V Stiff		110	3000	0
Transitional Beds (Fine) - Hard		110	4500	0
Whidbey Formation (Coarse)		135	0	38

1.034

250.00 lbs/ft2 250.00 lbs/ft2 250.00 lbs/ft2 250.00 lbs/ft2 250.00 lbs/ft2 250.00 lbs/ft2

Project	Edgewater Creek Bridge Replacement
Analysis Description	Figure C-3 - Cross Section A-A' - Global Slope Stability Analysis - L to R
Drawn By	SKS
Scale	1:700
Company	HWA GeoSciences, Inc.
Date	5/27/2020
Loading Scenario	Static Analysis - Existing Conditions - Ravine Slope

HWA GeoSciences Inc.
DBE/MWBE

SLIDEINTERPRET 8.029

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Abutment Walls		150		
Existing Fill		120	0	34
Colluvium (Fine)		105	475	0
Transitional Beds (Fine) - Stiff		110	1500	0
Transitional Beds (Fine) - V Stiff		110	3000	0
Transitional Beds (Fine) - Hard		110	4500	0
Whidbey Formation (Coarse)		135	0	38

Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+

1.577

250.00 lbs/ft2   250.00 lbs/ft2   250.00 lbs/ft2   250.00 lbs/ft2 25

W

Project		Edgewater Creek Bridge Replacement		
Analysis Description		Figure C-4 - Cross Section A-A' - Global Slope Stability Analysis - R to L		
Drawn By	SKS	Scale	1:600	Company
Date	5/27/2020			HWA GeoSciences, Inc.

Loading Scenario: Static Analysis - Existing Conditions - Ravine Slope

HWA GEOSCIENCES INC.
DBE/MWBE

SLIDEINTERPRET 8.029

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Abutment Walls		150		
Existing Fill		120	0	34
Colluvium (Fine)		105	475	0
Transitional Beds (Fine) - Stiff		110	1500	0
Transitional Beds (Fine) - V Stiff		110	3000	0
Transitional Beds (Fine) - Hard		110	4500	0
Whidbey Formation (Coarse)		135	0	38
Composite Material		135	17500	33

Safety Factor

0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+

1.808

Project		Edgewater Creek Bridge Replacement		
Analysis Description		Figure C-5 - Cross Section A-A' - Global Slope Stability Analysis - L to R		
Drawn By	SKS	Scale	1:600	Company
Date	5/27/2020			HWA GeoSciences, Inc.

HWA GEOSCIENCES INC.
DBE/MWBE

Loading Scenario: Static Analysis - Proposed Improvements

SLIDEINTERPRET 8.029

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Abutment Walls		150		
Existing Fill		120	0	34
Colluvium (Fine)		105	475	0
Transitional Beds (Fine) - Stiff		110	1500	0
Transitional Beds (Fine) - V Stiff		110	3000	0
Transitional Beds (Fine) - Hard		110	4500	0
Whidbey Formation (Coarse)		135	0	38
Composite Material		135	17500	33

Safety Factor: 0.000, 0.250, 0.500, 0.750, 1.000, 1.250, 1.500, 1.750, 2.000, 2.250, 2.500, 2.750, 3.000, 3.250, 3.500, 3.750, 4.000, 4.250, 4.500, 4.750, 5.000, 5.250, 5.500, 5.750, 6.000+

2.453

250.00 lbs/ft2   250.00 lbs/ft2   250.00 lbs/ft2   250.00 lbs/ft2   250.00 lbs/ft2

Project		Edgewater Creek Bridge Replacement			
Analysis Description		Figure C-6 - Cross Section A-A' - Global Slope Stability Analysis - R to L			
Drawn By	SKS	**Scale**	1:600	**Company**	HWA GeoSciences, Inc.
Date	5/27/2020			**Loading Scenario**	Static Analysis - Proposed Improvements

HWA GeoSciences Inc.
DBE/MWBE

SLIDEINTERPRET 8.029

Safety Factor

	0.000
	0.250
	0.500
	0.750
	1.000
	1.250
	1.500
	1.750
	2.000
	2.250
	2.500
	2.750
	3.000
	3.250
	3.500
	3.750
	4.000
	4.250
	4.500
	4.750
	5.000
	5.250
	5.500
	5.750
	6.000+

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Abutment Walls		150		
Existing Fill		120	0	34
Colluvium (Fine)		105	475	0
Transitional Beds (Fine) - Stiff		110	1500	0
Transitional Beds (Fine) - V Stiff		110	3000	0
Transitional Beds (Fine) - Hard		110	4500	0
Whidbey Formation (Coarse)		135	0	38
Composite Material		135	17500	33

1.676

Project	Edgewater Creek Bridge Replacement		
Analysis Description	Figure C-7 - Cross Section A-A' - Global Slope Stability Analysis - L to R		
Drawn By	SKS	Scale 1:700	Company HWA GeoSciences, Inc.
Date	5/27/2020	Loading Scenario	Static Analysis - Proposed Improvements - Ravine Slope

GEOSCIENCES INC.
DBE/MWBE

SLIDEINTERPRET 8.029

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Abutment Walls		150		
Existing Fill		120	0	34
Colluvium (Fine)		105	475	0
Transitional Beds (Fine) - Stiff		110	1500	0
Transitional Beds (Fine) - V.Stiff		110	3000	0
Transitional Beds (Fine) - Hard		110	4500	0
Whidbey Formation (Coarse)		135	0	38
Composite Material		135	17500	33

Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+

1.656

250.00 lbs/ft2   250.00 lbs/ft2 250.00 lbs/ft2 250.00 lbs/ft2 250.00 lbs/ft2

W

Project	Edgewater Creek Bridge Replacement		
Analysis Description	Figure C-8 - Cross Section A-A' - Global Slope Stability Analysis - R to L		
Drawn By	SKS	Scale 1:600	Company HWA GeoSciences, Inc.
Date	5/27/2020	Loading Scenario	Static Analysis - Proposed Improvements - Ravine Slope

Safety Factor

	0.000
	0.250
	0.500
	0.750
	1.000
	1.250
	1.500
	1.750
	2.000
	2.250
	2.500
	2.750
	3.000
	3.250
	3.500
	3.750
	4.000
	4.250
	4.500
	4.750
	5.000
	5.250
	5.500
	5.750
	6.000+

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Abutment Walls		150		
Existing Fill		120	0	34
Colluvium (Fine)		105	475	0
Transitional Beds (Fine) - Stiff		110	1500	0
Transitional Beds (Fine) - V Stiff		110	3000	0
Transitional Beds (Fine) - Hard		110	4500	0
Whidbey Formation (Coarse)		135	0	38

0.847

► 0.242

W

Project	Edgewater Creek Bridge Replacement			
Analysis Description	Figure C-9 - Cross Section A-A' - Global Slope Stability Analysis - L to R			
Drawn By	SKS	Scale	1:600	Company
Date	5/27/2020	Loading Scenario	Pseudostatic Analysis - Existing Conditions	HWA GeoSciences, Inc.

HWA GEOSCIENCES INC.
DBE/MWBE

SLIDEINTERPRET 8.029

Safety Factor

	0.000
	0.250
	0.500
	0.750
	1.000
	1.250
	1.500
	1.750
	2.000
	2.250
	2.500
	2.750
	3.000
	3.250
	3.500
	3.750
	4.000
	4.250
	4.500
	4.750
	5.000
	5.250
	5.500
	5.750
	6.000+

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Abutment Walls		150		
Existing Fill		120	0	34
Colluvium (Fine)		105	475	0
Transitional Beds (Fine) - Stiff		110	1500	0
Transitional Beds (Fine) - V. Stiff		110	3000	0
Transitional Beds (Fine) - Hard		110	4500	0
Whidbey Formation (Coarse)		135	0	38

0.913

◄ 0.242

W

Project		Edgewater Creek Bridge Replacement
Analysis Description		Figure C-10 - Cross Section A-A' - Global Slope Stability Analysis - R to L
Drawn By	SKS	
Scale	1:600	Company
Date	5/27/2020	Loading Scenario

HWA GeoSciences, Inc.

Pseudostatic Analysis - Existing Conditions

HWA GeoSciences Inc. DBE/MWBE

SLIDEINTERPRET 8.029

Safety Factor

	0.000
	0.250
	0.500
	0.750
	1.000
	1.250
	1.500
	1.750
	2.000
	2.250
	2.500
	2.750
	3.000
	3.250
	3.500
	3.750
	4.000
	4.250
	4.500
	4.750
	5.000
	5.250
	5.500
	5.750
	6.000+

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Abutment Walls		150		
Existing Fill		120	0	34
Colluvium (Fine)		105	475	0
Transitional Beds (Fine) - Stiff		110	1500	0
Transitional Beds (Fine) - V Stiff		110	3000	0
Transitional Beds (Fine) - Hard		110	4500	0
Whidbey Formation (Coarse)		135	0	38

▶ 0.242

0.704

W

	Project	Edgewater Creek Bridge Replacement
	Analysis Description	Figure C-11 - Cross Section A-A' - Global Slope Stability Analysis - L to R
	Drawn By	SKS
	Scale	1:600
	Company	HWA GeoSciences, Inc.
	Date	5/27/2020
	Loading Scenario	Pseudostatic Analysis - Existing Conditions - Ravine Slope

HWA GeoSciences Inc.
DBE/MWBE

SLIDEINTERPRET 8.029

Safety Factor

0.000	
0.250	
0.500	
0.750	
1.000	
1.250	
1.500	
1.750	
2.000	
2.250	
2.500	
2.750	
3.000	
3.250	
3.500	
3.750	
4.000	
4.250	
4.500	
4.750	
5.000	
5.250	
5.500	
5.750	
6.000+	

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Abutment Walls		150		
Existing Fill		120	0	34
Colluvium (Fine)		105	475	0
Transitional Beds (Fine) - Stiff		110	1500	0
Transitional Beds (Fine) - V Stiff		110	3000	0
Transitional Beds (Fine) - Hard		110	4500	0
Whidbey Formation (Coarse)		135	0	38

◄ 0.242

1.094

W

Safety Factor	
	0.000
	0.250
	0.500
	0.750
	1.000
	1.250
	1.500
	1.750
	2.000
	2.250
	2.500
	2.750
	3.000
	3.250
	3.500
	3.750
	4.000
	4.250
	4.500
	4.750
	5.000
	5.250
	5.500
	5.750
	6.000+

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Abutment Walls		150		
Existing Fill		120	0	34
Colluvium (Fine)		105	475	0
Transitional Beds (Fine) - Stiff		110	1500	0
Transitional Beds (Fine) - V. Stiff		110	3000	0
Transitional Beds (Fine) - Hard		110	4500	0
Whidbey Formation (Coarse)		135	0	38
Composite Material		135	17500	33

▶ 0.242

1.304

W

	Project	Edgewater Creek Bridge Replacement		
HWA GEOSCIENCES INC. DBE/MWBE	Analysis Description	Figure C-13 - Cross Section A-A' - Global Slope Stability Analysis - L to R		
	Drawn By SKS	Scale 1:600	Company HWA GeoSciences, Inc.	
	Date 5/27/2020	Loading Scenario Pseudostatic Analysis - Proposed Improvements		

SLIDEINTERPRET 8.029

Safety Factor	
	0.000
	0.250
	0.500
	0.750
	1.000
	1.250
	1.500
	1.750
	2.000
	2.250
	2.500
	2.750
	3.000
	3.250
	3.500
	3.750
	4.000
	4.250
	4.500
	4.750
	5.000
	5.250
	5.500
	5.750
	6.000+

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Abutment Walls		150		
Existing Fill		120	0	34
Colluvium (Fine)		105	475	0
Transitional Beds (Fine) - Stiff		110	1500	0
Transitional Beds (Fine) - V Stiff		110	3000	0
Transitional Beds (Fine) - Hard		110	4500	0
Whidbey Formation (Coarse)		135	0	38
Composite Material		135	17500	33

1.643

◄ 0.242

W

	Project	Edgewater Creek Bridge Replacement			
	Analysis Description	Figure C-14 - Cross Section A-A' - Global Slope Stability Analysis - R to L			
	Drawn By	SKS	Scale	1:700	Company	HWA GeoSciences, Inc.
	Date	5/27/2020	Loading Scenario	Pseudostatic Analysis - Proposed Improvements	

HWA GeoSciences Inc.
DBE/MWBE

SLIDEINTERPRET 8.029

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Abutment Walls		150		
Existing Fill		120	0	34
Colluvium (Fine)		105	475	0
Transitional Beds (Fine) - Stiff		110	1500	0
Transitional Beds (Fine) - V Stiff		110	3000	0
Transitional Beds (Fine) - Hard		110	4500	0
Whidbey Formation (Coarse)		135	0	38
Composite Material		135	17500	33

Safety Factor

0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+

► 0.242

1.115

W

	Project	Edgewater Creek Bridge Replacement			
HWA GeoSciences Inc. DBE/MWBE	Analysis Description	Figure C-15 - Cross Section A-A' - Global Slope Stability Analysis - L to R			
	Drawn By SKS		Scale 1:600	Company	HWA GeoSciences, Inc.
	Date 5/27/2020			Loading Scenario Pseudostatic Analysis - Proposed Improvements - Ravine Slope	

SLIDEINTERPRET 8.029

Figure C-16 - Cross Section A-A' - Global Slope Stability Analysis - R to L

Safety Factor	
0.000	(red)
0.250	
0.500	
0.750	
1.000	
1.250	
1.500	
1.750	
2.000	
2.250	
2.500	
2.750	
3.000	
3.250	
3.500	
3.750	
4.000	
4.250	
4.500	
4.750	
5.000	
5.250	
5.500	
5.750	
6.000+	(blue)

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Abutment Walls		150		
Existing Fill		120	0	34
Colluvium (Fine)		105	475	0
Transitional Beds (Fine) - Stiff		110	1500	0
Transitional Beds (Fine) - V Stiff		110	3000	0
Transitional Beds (Fine) - Hard		110	4500	0
Whidbey Formation (Coarse)		135	0	38
Composite Material		135	17500	33

0.242

1.116

W

Project	Edgewater Creek Bridge Replacement		
Analysis Description	Figure C-16 - Cross Section A-A' - Global Slope Stability Analysis - R to L		
Drawn By	SKS	**Scale** 1:600	**Company** HWA GeoSciences, Inc.
Date	5/27/2020		**Loading Scenario** Pseudostatic Analysis - Proposed Improvements - Ravine Slope

HWA GEOSCIENCES INC.
DBE/MWBE

SLIDEINTERPRET 8.029

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Abutment Walls		150		
Existing Fill		120	0	34
Colluvium (Fine)		105	475	0
Transitional Beds (Fine) - Stiff		110	1500	0
Transitional Beds (Fine) - V Stiff		110	3000	0
Transitional Beds (Fine) - Hard		110	4500	0
Whidbey Formation (Coarse)		135	0	38
Colluvium Post LQ		105	0	8

Safety Factor

0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+

1:533

lbs/ft2 250.00 lbs/ft2 250.00 lbs/ft2 250.00 lbs/ft2 250.00 lbs/ft2 250.00 lbs/ft2

W

Project		Edgewater Creek Bridge Replacement		
Analysis Description		Figure C-17 - Cross Section A-A' - Global Slope Stability Analysis - L to R		
Drawn By	SKS	Scale	1:600	Company
				HWA GeoSciences, Inc.
Date	5/27/2020	Loading Scenario		Post LQ Analysis - Existing Conditions

HWA GEOSCIENCES INC.
DBE/MWBE

SLIDEINTERPRET 8.029

Safety Factor	
0.000	
0.250	
0.500	
0.750	
1.000	
1.250	
1.500	
1.750	
2.000	
2.250	
2.500	
2.750	
3.000	
3.250	
3.500	
3.750	
4.000	
4.250	
4.500	
4.750	
5.000	
5.250	
5.500	
5.750	
6.000+	

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Abutment Walls		150		
Existing Fill		120	0	34
Colluvium (Fine)		105	475	0
Transitional Beds (Fine) - Stiff		110	1500	0
Transitional Beds (Fine) - V Stiff		110	3000	0
Transitional Beds (Fine) - Hard		110	4500	0
Whidbey Formation (Coarse)		135	0	38
Colluvium Post LQ		105	0	8

0.803

SLIDEINTERPRET 8.029

Project	Edgewater Creek Bridge Replacement
Analysis Description	Figure C-18 - Cross Section A-A' - Global Slope Stability Analysis - L to R
Drawn By	SKS
Scale	1:600
Company	HWA GeoSciences, Inc.
Date	5/27/2020
Loading Scenario	Post LQ Analysis - Ravine Slope

HWA GeoSciences Inc.
DBE/MWBE

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Structural Fill		140	0	38
Abutment Walls		150		
Existing Fill		120	0	34
Colluvium (Fine)		105	475	0
Transitional Beds (Fine) - Stiff		110	1500	0
Transitional Beds (Fine) - V Stiff		110	3000	0
Transitional Beds (Fine) - Hard		110	4500	0
Whidbey Formation (Coarse)		135	0	38
Colluvium Post LQ		105	0	8

Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+

1.511

Project	Edgewater Creek Bridge Replacement
Analysis Description	Figure C-19 - Cross Section A-A' - Global Slope Stability Analysis - L to R
Drawn By SKS	**Scale** 1:600 **Company** HWA GeoSciences, Inc.
Date 5/27/2020	**Loading Scenario** Post LQ Analysis - Ravine Slope Improvements

HWA GEOSCIENCES INC.
DBE/MWBE

SLIDEINTERPRET 8.029

Figure C-20 - Cross Section A-A' - Global Slope Stability Analysis - L to R

Safety Factor legend:
0.000, 0.250, 0.500, 0.750, 1.000, 1.250, 1.500, 1.750, 2.000, 2.250, 2.500, 2.750, 3.000, 3.250, 3.500, 3.750, 4.000, 4.250, 4.500, 4.750, 5.000, 5.250, 5.500, 5.750, 6.000+

3.419

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Abutment Walls		150		
Existing Fill		120	0	34
Colluvium (Fine)		105	475	0
Transitional Beds (Fine) - Stiff		110	1500	0
Transitional Beds (Fine) - V Stiff		110	3000	0
Transitional Beds (Fine) - Hard		110	4500	0
Whidbey Formation (Coarse)		135	0	38
Composite Material		135	17500	33
Colluvium Post LQ		105	0	8

W

Project	Edgewater Creek Bridge Replacement			
Analysis Description	Figure C-20 - Cross Section A-A' - Global Slope Stability Analysis - L to R			
Drawn By	SKS	Scale	1:600	Company	HWA GeoSciences, Inc.
Date	5/27/2020	Loading Scenario	Post LQ Analysis - Proposed Improvements	

HWA GeoSciences Inc.
DBE/MWBE

SLIDEINTERPRET 8.029

Safety Factor

	0.000
	0.250
	0.500
	0.750
	1.000
	1.250
	1.500
	1.750
	2.000
	2.250
	2.500
	2.750
	3.000
	3.250
	3.500
	3.750
	4.000
	4.250
	4.500
	4.750
	5.000
	5.250
	5.500
	5.750
	6.000+

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Abutment Walls		150		
Existing Fill		120	0	34
Colluvium (Fine)		105	475	0
Transitional Beds (Fine) - Stiff		110	1500	0
Transitional Beds (Fine) - V Stiff		110	3000	0
Transitional Beds (Fine) - Hard		110	4500	0
Whidbey Formation (Coarse)		135	0	38
Composite Material		135	17500	33
Colluvium Post LQ		105	0	8

1.156

SLIDEINTERPRET 8.029

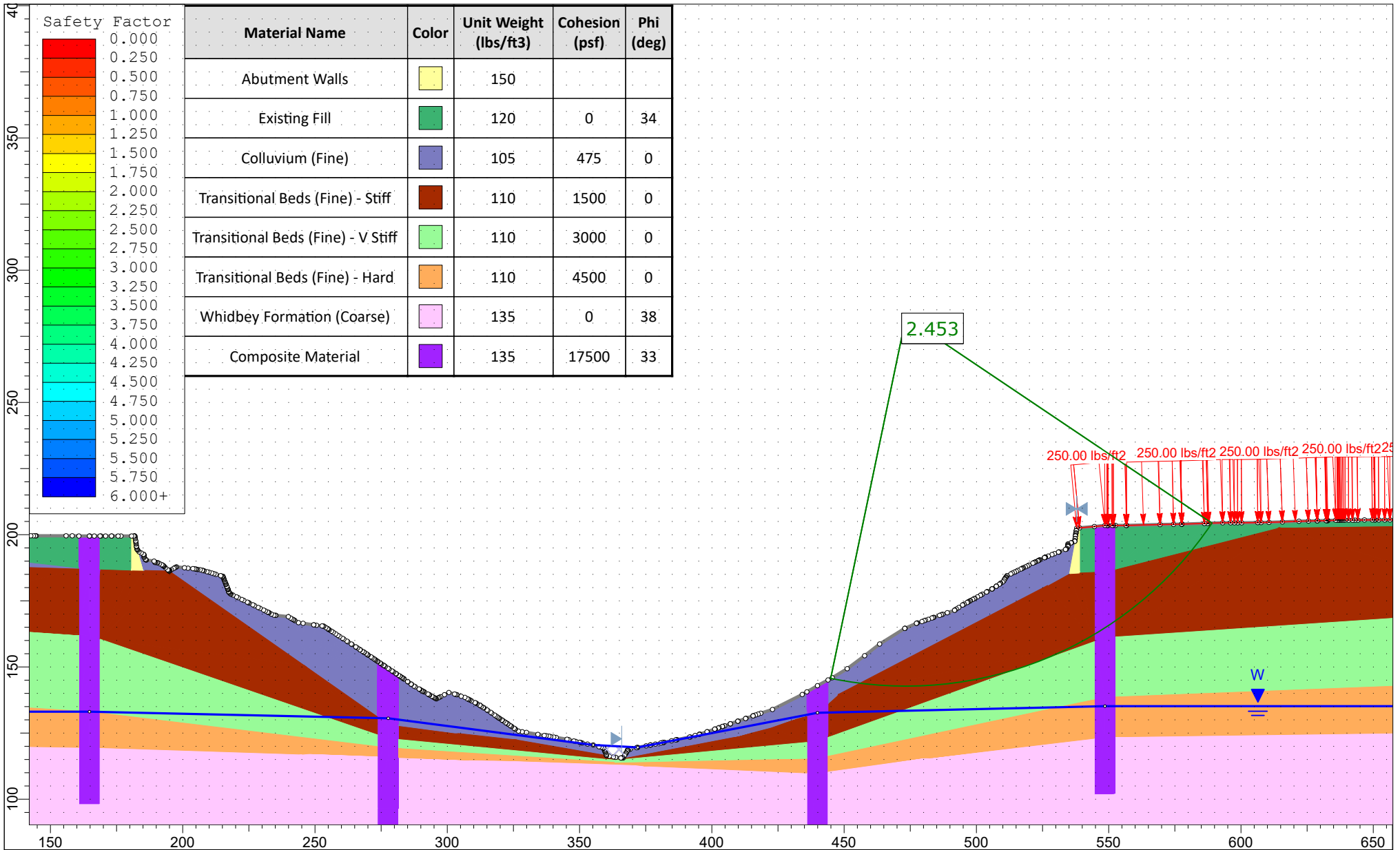|	
Project	Edgewater Creek Bridge Replacement
Analysis Description	Figure C-21 - Cross Section A-A' - Global Slope Stability Analysis - L to R
Drawn By	SKS
Scale	1:600
Company	HWA GeoSciences, Inc.
Date	5/27/2020
Loading Scenario	Post LQ Analysis - Proposed Improvements - Ravine Slope

HWA GEOSCIENCES INC.
DBE/MWBE

APPENDIX D

CROSS SECTION A-A' (NORTH SLOPE) GLOBAL SLOPE STABILITY ANALYSIS

Safety Factor

	0.000
	0.250
	0.500
	0.750
	1.000
	1.250
	1.500
	1.750
	2.000
	2.250
	2.500
	2.750
	3.000
	3.250
	3.500
	3.750
	4.000
	4.250
	4.500
	4.750
	5.000
	5.250
	5.500
	5.750
	6.000+

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Abutment Walls		150		
Existing Fill		120	0	34
Colluvium (Fine)		105	475	0
Transitional Beds (Fine) - Stiff		110	2000	0
Transitional Beds (Fine) - V Stiff		110	4000	0
Transitional Beds (Fine) - Hard		110	5500	0
Whidbey Formation (Coarse)		135	0	38

1.947

250.00 lbs/ft2   250.00 lbs/ft2   250.00 lbs/ft2   250.00 lbs/ft2

Project	Edgewater Creek Bridge Replacement			
Analysis Description	Figure D-1 - Cross Section B-B' - Global Slope Stability Analysis - L to R			
Drawn By	SKS	Scale	1:500	Company	HWA GeoSciences, Inc.
Date	10/9/2020	Loading Scenario		Static Analysis - Existing Conditions

SLIDEINTERPRET 8.029

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Existing Fill		120	0	34
Colluvium (Fine)		105	475	0
Transitional Beds (Fine) - Stiff		110	2000	0
Transitional Beds (Fine) - V Stiff		110	4000	0
Transitional Beds (Fine) - Hard		110	5500	0
Whidbey Formation (Coarse)		135	0	38

Safety Factor

0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+

2.145

250.00 lbs/ft2   250.00 lbs/ft2 250.00 lbs/ft2 250.00 lbs/ft2

W

W

Project	Edgewater Creek Bridge Replacement		
Analysis Description	Figure D-2 - Cross Section B-B' - Global Slope Stability Analysis - L to R		
Drawn By	SKS	Scale 1:500	Company HWA GeoSciences, Inc.
Date	10/9/2020	Loading Scenario	Static Analysis - Existing Conditions - Ravine Slope

HWA GeoSciences Inc.
DBE/MWBE

SLIDEINTERPRET 8.029

Safety Factor	
0.000	
0.250	
0.500	
0.750	
1.000	
1.250	
1.500	
1.750	
2.000	
2.250	
2.500	
2.750	
3.000	
3.250	
3.500	
3.750	
4.000	
4.250	
4.500	
4.750	
5.000	
5.250	
5.500	
5.750	
6.000+	

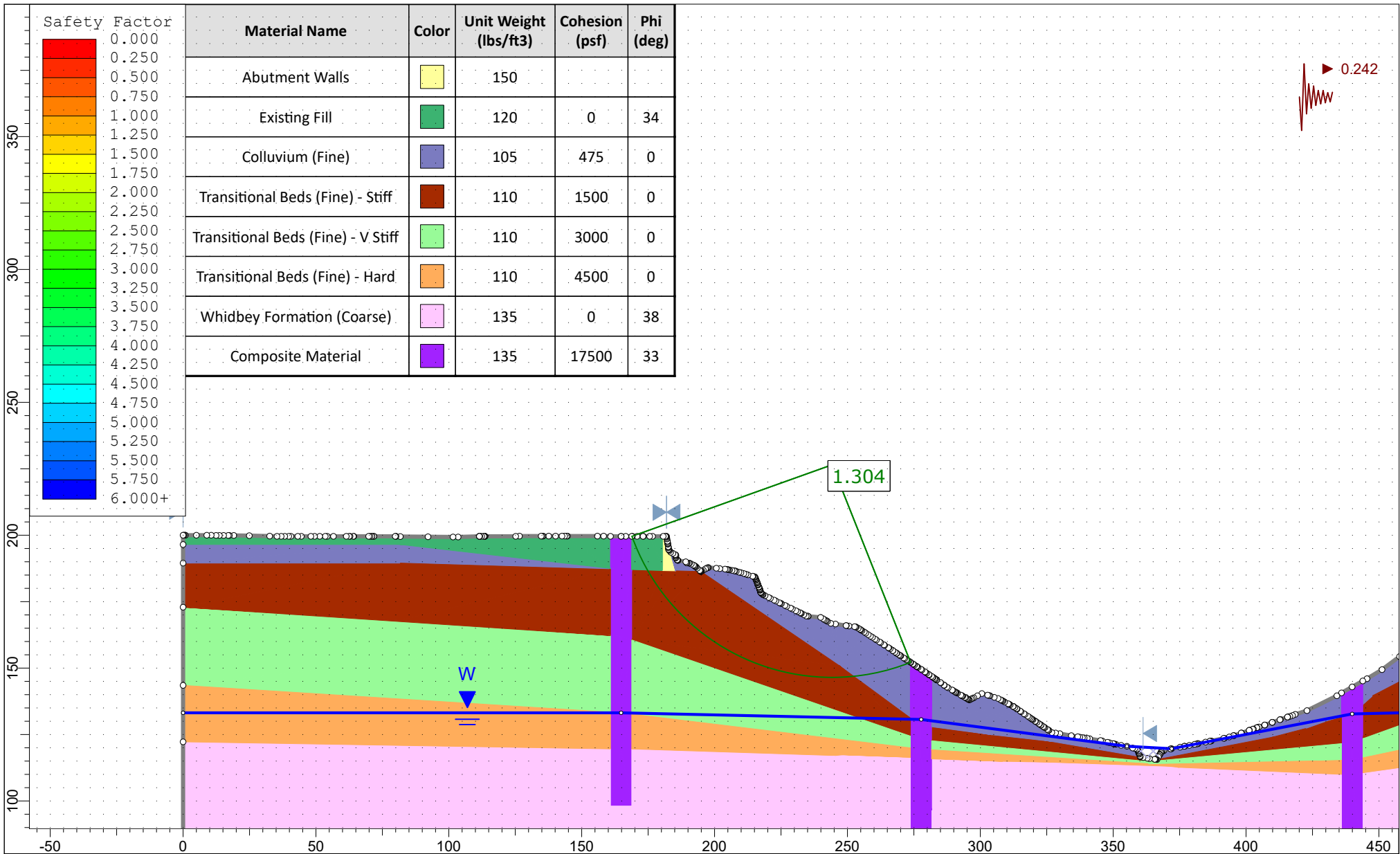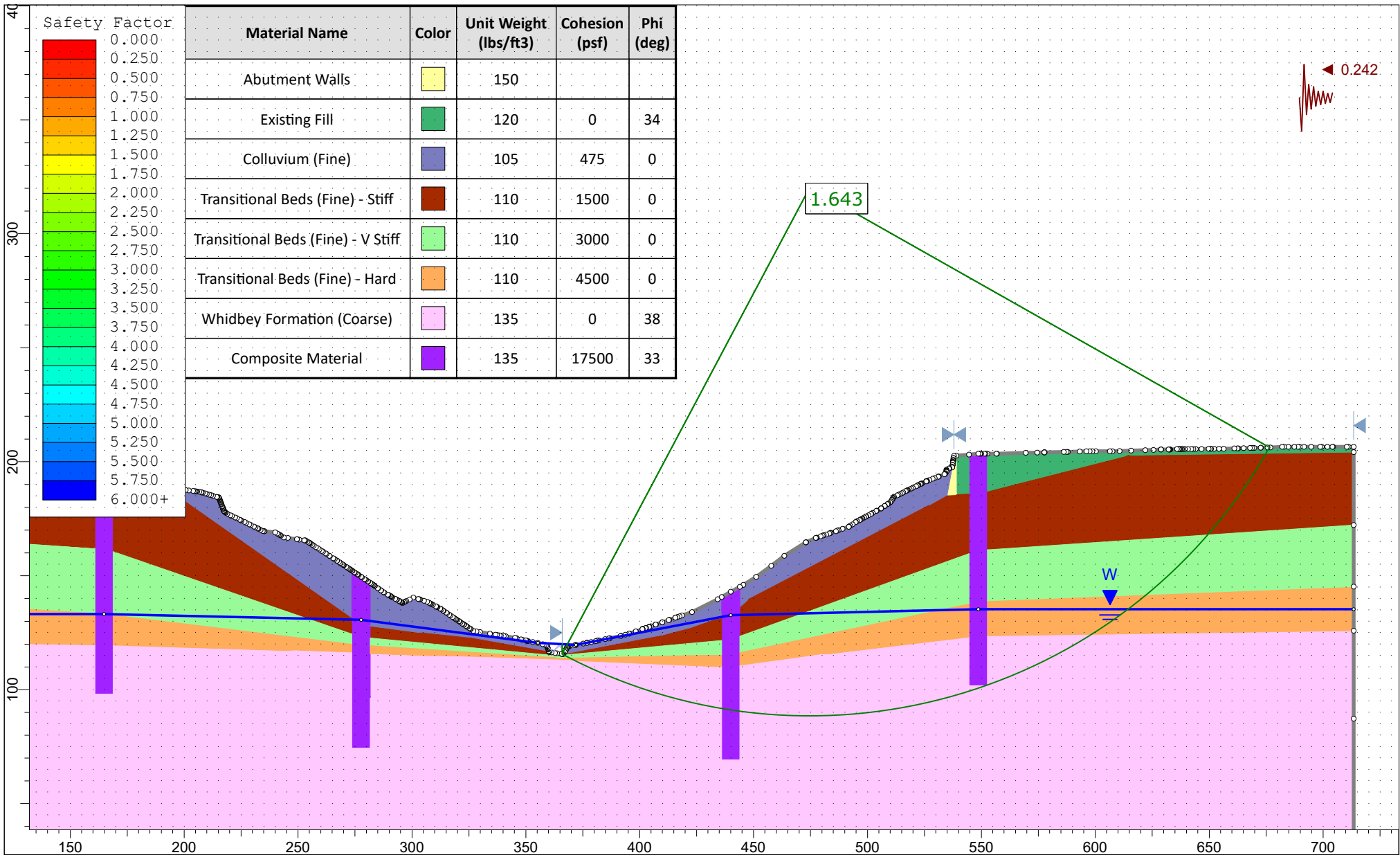| Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Existing Fill		120	0	34
Colluvium (Fine)		105	475	0
Transitional Beds (Fine) - Stiff		110	2000	0
Transitional Beds (Fine) - V Stiff		110	4000	0
Transitional Beds (Fine) - Hard		110	5500	0
Whidbey Formation (Coarse)		135	0	38

► 0.242

1.312

Project	Edgewater Creek Bridge Replacement
Analysis Description	Figure D-3 - Cross Section B-B' - Global Slope Stability Analysis - L to R
Drawn By	SKS
Scale	1:500
Company	HWA GeoSciences, Inc.
Date	10/9/2020
Loading Scenario	Pseudostatic Analysis - Existing Conditions

HWA GEOSCIENCES INC.
DBE/MWBE

SLIDEINTERPRET 8.029

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Existing Fill		120	0	34
Colluvium (Fine)		105	475	0
Transitional Beds (Fine) - Stiff		110	2000	0
Transitional Beds (Fine) - V Stiff		110	4000	0
Transitional Beds (Fine) - Hard		110	5500	0
Whidbey Formation (Coarse)		135	0	38
Composite Material		135	17500	33

Safety Factor: 0.000, 0.250, 0.500, 0.750, 1.000, 1.250, 1.500, 1.750, 2.000, 2.250, 2.500, 2.750, 3.000, 3.250, 3.500, 3.750, 4.000, 4.250, 4.500, 4.750, 5.000, 5.250, 5.500, 5.750, 6.000+

1.586

▶ 0.242

Project	Edgewater Creek Bridge Replacement
Analysis Description	Figure D-4 - Cross Section B-B' - Global Slope Stability Analysis - L to R

Drawn By	SKS	Scale	1:600	Company	HWA GeoSciences, Inc.
Date	10/9/2020			Loading Scenario	Pseudostatic Analysis - Proposed Conditions

HWA GEOSCIENCES INC.
DBE/MWBE

SLIDEINTERPRET 8.029

APPENDIX E

CROSS SECTION B-B' (CRITICAL SLOPE) GLOBAL SLOPE STABILITY ANALYSIS

Safety Factor

	0.000
	0.250
	0.500
	0.750
	1.000
	1.250
	1.500
	1.750
	2.000
	2.250
	2.500
	2.750
	3.000
	3.250
	3.500
	3.750
	4.000
	4.250
	4.500
	4.750
	5.000
	5.250
	5.500
	5.750
	6.000+

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Abutment Walls		150		
Existing Fill		120	0	34
Colluvium (Fine)		105	475	0
Transitional Beds (Fine) - Stiff		110	1500	0
Transitional Beds (Fine) - V Stiff		110	3000	0
Transitional Beds (Fine) - Hard		110	4500	0
Whidbey Formation (Coarse)		135	0	38

1.108

250.00 lbs/ft2   250.00 lbs/ft2   250.00 lbs/ft2

36.650

W

W

SLIDEINTERPRET 8.029

Project	Edgewater Creek Bridge Replacement			
Analysis Description	Figure E-1 - Cross Section C-C' - Global Slope Stability Analysis - L to R			
Drawn By	SKS	Scale	1:500	Company	HWA GeoSciences, Inc.
Date	10/9/2020	Loading Scenario	Static Analysis - Existing Conditions	

HWA | GEOSCIENCES INC.
DBE/MWBE

Safety Factor

	0.000
	0.250
	0.500
	0.750
	1.000
	1.250
	1.500
	1.750
	2.000
	2.250
	2.500
	2.750
	3.000
	3.250
	3.500
	3.750
	4.000
	4.250
	4.500
	4.750
	5.000
	5.250
	5.500
	5.750
	6.000+

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Abutment Walls		150		
Existing Fill		120	0	34
Colluvium (Fine)		105	475	0
Transitional Beds (Fine) - Stiff		110	1500	0
Transitional Beds (Fine) - V Stiff		110	3000	0

250.00 lbs/ft2   250.00 lbs/ft2 250.00 lbs/ft2

1.943

W   W

Project	Edgewater Creek Bridge Replacement		
Analysis Description	Figure E-2 - Cross Section C-C' - Global Slope Stability Analysis - L to R		
Drawn By	SKS	Scale 1:500	Company HWA GeoSciences, Inc.
Date	10/9/2020	Loading Scenario	Static Analysis - Existing Conditions - Ravine Slope

HWA GEOSCIENCES INC.
DBE/MWBE

SLIDEINTERPRET 8.029

Safety Factor	
0.000	
0.250	
0.500	
0.750	
1.000	
1.250	
1.500	
1.750	
2.000	
2.250	
2.500	
2.750	
3.000	
3.250	
3.500	
3.750	
4.000	
4.250	
4.500	
4.750	
5.000	
5.250	
5.500	
5.750	
6.000+	

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Abutment Walls		150		
Existing Fill		120	0	34
Colluvium (Fine)		105	475	0
Transitional Beds (Fine) - Stiff		110	1500	0
Transitional Beds (Fine) - V Stiff		110	3000	0
Transitional Beds (Fine) - Hard		110	4500	0
Whidbey Formation (Coarse)		135	0	38
Composite Material		135	17500	33

250.00 lbs/ft2   250.00 lbs/ft2 250.00 lbs/ft2

1.292

W

W

	Project		Edgewater Creek Bridge Replacement		
	Analysis Description		Figure E-3 - Cross Section C-C' - Global Slope Stability Analysis - L to R		
	Drawn By	SKS	Scale	1:500	Company	HWA GeoSciences, Inc.
	Date	10/9/2020	Loading Scenario		Static Analysis - Proposed Conditions

HWA GEOSCIENCES INC.
DBE/MWBE

SLIDEINTERPRET 8.029

Safety Factor	
	0.000
	0.250
	0.500
	0.750
	1.000
	1.250
	1.500
	1.750
	2.000
	2.250
	2.500
	2.750
	3.000
	3.250
	3.500
	3.750
	4.000
	4.250
	4.500
	4.750
	5.000
	5.250
	5.500
	5.750
	6.000+

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Abutment Walls		150		
Existing Fill		120	0	34
Colluvium (Fine)		105	475	0
Transitional Beds (Fine) - Stiff		110	1500	0
Transitional Beds (Fine) - V Stiff		110	3000	0
Transitional Beds (Fine) - Hard		110	4500	0
Whidbey Formation (Coarse)		135	0	38
Composite Material		135	17500	33

250.00 lbs/ft2   250.00 lbs/ft2   250.00 lbs/ft2

1.944

47.893

W

W

HWA GEOSCIENCES INC.
DBE/MWBE

Project	Edgewater Creek Bridge Replacement		
Analysis Description	Figure E-4 - Cross Section C-C' - Global Slope Stability Analysis - L to R		
Drawn By	SKS	Scale 1:500	Company HWA GeoSciences, Inc.
Date	10/9/2020	Loading Scenario Static Analysis - Proposed Conditions - Ravine Slope	

Figure E-5 - Cross Section C-C' - Global Slope Stability Analysis - L to R

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Abutment Walls		150		
Existing Fill		120	0	34
Colluvium (Fine)		105	475	0
Transitional Beds (Fine) - Stiff		110	1500	0
Transitional Beds (Fine) - V Stiff		110	3000	0
Transitional Beds (Fine) - Hard		110	4500	0
Whidbey Formation (Coarse)		135	0	38

Safety Factor

0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+

► 0.242

0.868

	Project	Edgewater Creek Bridge Replacement		
	Analysis Description	Figure E-5 - Cross Section C-C' - Global Slope Stability Analysis - L to R		
	Drawn By SKS	Scale 1:500	Company	HWA GeoSciences, Inc.
	Date 10/9/2020		Loading Scenario	Pseudostatic Analysis - Existing Conditions

SLIDEINTERPRET 8.029

HWA GeoSciences Inc.
DBE/MWBE

Figure E-6 - Cross Section C-C' - Global Slope Stability Analysis - L to R

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Abutment Walls		150		
Existing Fill		120	0	34
Colluvium (Fine)		105	475	0
Transitional Beds (Fine) - Stiff		110	1500	0
Transitional Beds (Fine) - V Stiff		110	3000	0
Transitional Beds (Fine) - Hard		110	4500	0
Whidbey Formation (Coarse)		135	0	38

Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+

► 0.242

1.393

Project				
	Edgewater Creek Bridge Replacement			
Analysis Description	Figure E-6 - Cross Section C-C' - Global Slope Stability Analysis - L to R			
Drawn By	SKS	Scale 1:500	Company	HWA GeoSciences, Inc.
Date	10/9/2020	Loading Scenario	Pseudostatic Analysis - Existing Conditions - Ravine Slope	

HWA GEOSCIENCES INC.
DBE/MWBE

SLIDEINTERPRET 8.029

Safety Factor

0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Abutment Walls		150		
Existing Fill		120	0	34
Colluvium (Fine)		105	475	0
Transitional Beds (Fine) - Stiff		110	1500	0
Transitional Beds (Fine) - V Stiff		110	3000	0
Transitional Beds (Fine) - Hard		110	4500	0
Whidbey Formation (Coarse)		135	0	38
Composite Material		135	17500	33

▶ 0.242

1.019

W

W

Project	Edgewater Creek Bridge Replacement
Analysis Description	Figure E-7 - Cross Section C-C' - Global Slope Stability Analysis - L to R
Drawn By	SKS
Scale	1:500
Company	HWA GeoSciences, Inc.
Date	10/9/2020
Loading Scenario	Pseudostatic Analysis - Proposed Conditions

HWA GEOSCIENCES INC.
DBE/MWBE

SLIDEINTERPRET 8.029

Safety Factor

	0.000
	0.250
	0.500
	0.750
	1.000
	1.250
	1.500
	1.750
	2.000
	2.250
	2.500
	2.750
	3.000
	3.250
	3.500
	3.750
	4.000
	4.250
	4.500
	4.750
	5.000
	5.250
	5.500
	5.750
	6.000+

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Abutment Walls		150		
Existing Fill		120	0	34
Colluvium (Fine)		105	475	0
Transitional Beds (Fine) - Stiff		110	1500	0
Transitional Beds (Fine) - V Stiff		110	3000	0
Transitional Beds (Fine) - Hard		110	4500	0
Whidbey Formation (Coarse)		135	0	38
Composite Material		135	17500	33

► 0.242

1.407

W

W

Project	Edgewater Creek Bridge Replacement
Analysis Description	Figure E-8 - Cross Section C-C' - Global Slope Stability Analysis - L to R
Drawn By	SKS
Scale	1:500
Company	HWA GeoSciences, Inc.
Date	10/9/2020
Loading Scenario	Pseudostatic Analysis - Proposed Conditions - Ravine Slope

HWA GEOSCIENCES INC.
DBE/MWBE

SLIDEINTERPRET 8.029

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Abutment Walls		150		
Existing Fill		120	0	34
Colluvium (Fine)		105	475	0
Transitional Beds (Fine) - Stiff		110	1500	0
Transitional Beds (Fine) - V Stiff		110	3000	0
Transitional Beds (Fine) - Hard		110	4500	0
Whidbey Formation (Coarse)		135	0	38
Composite Material		135	17500	33

Safety Factor

0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
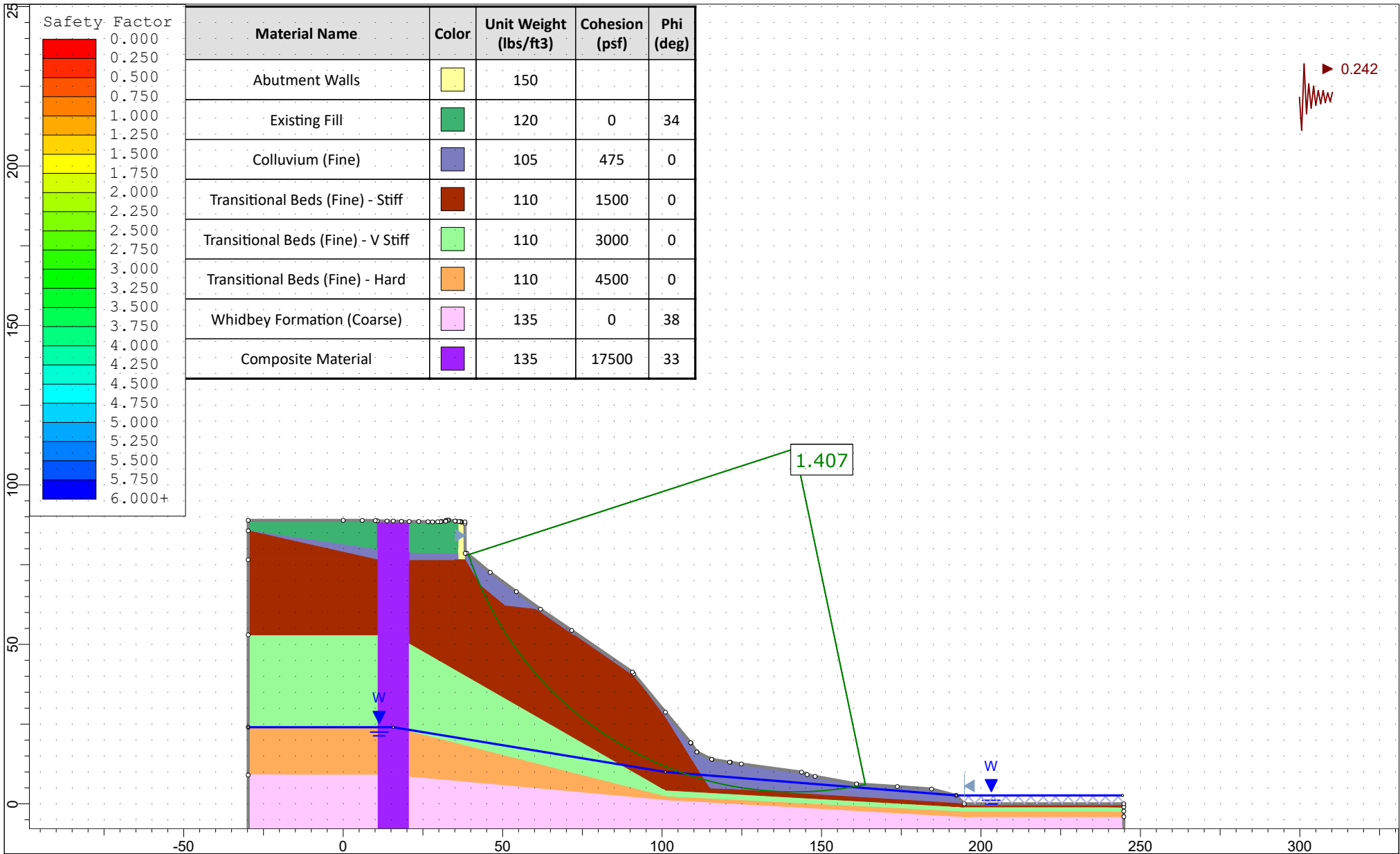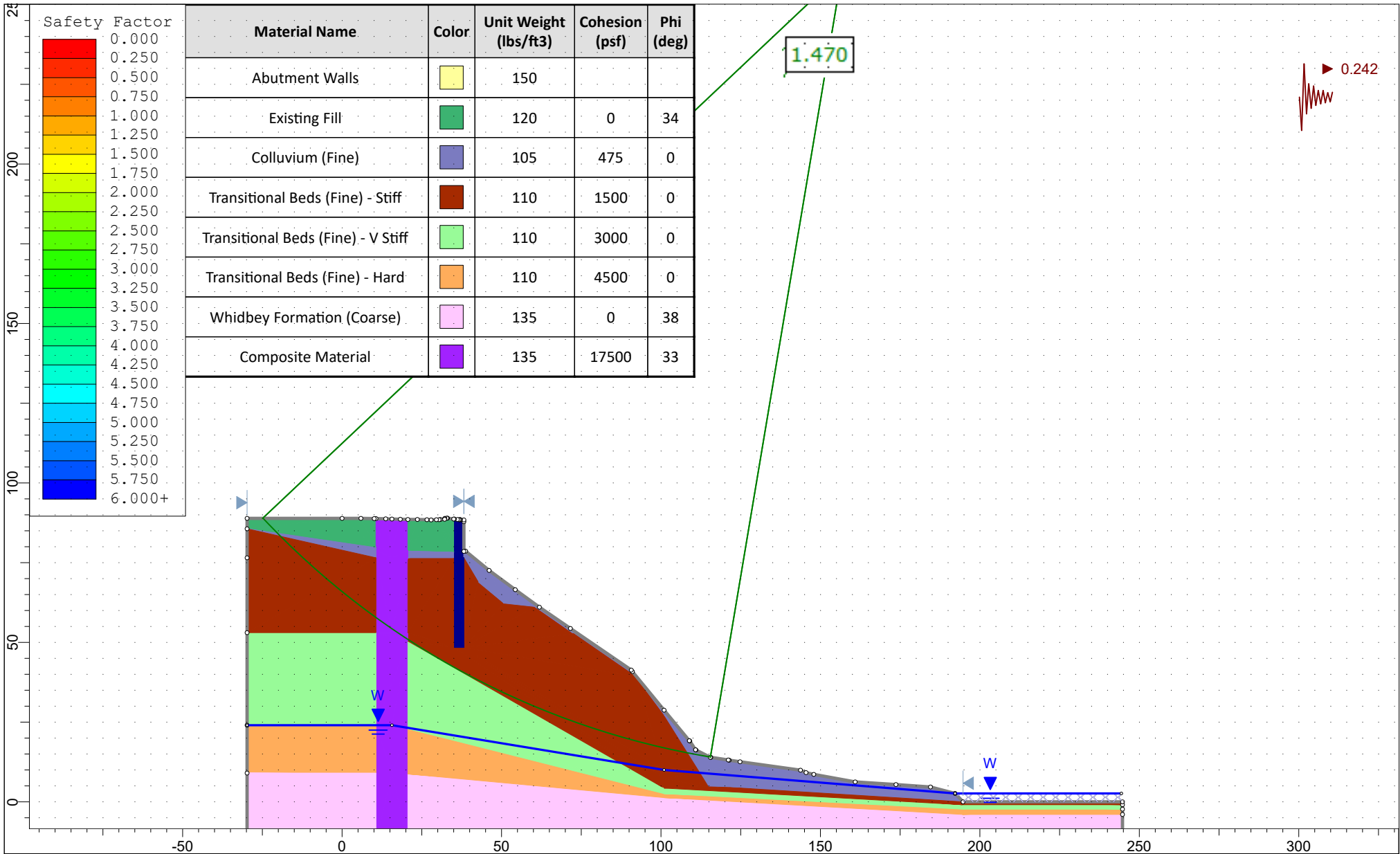4.500
4.750
5.000
5.250
5.500
5.750
6.000+

1.470

► 0.242

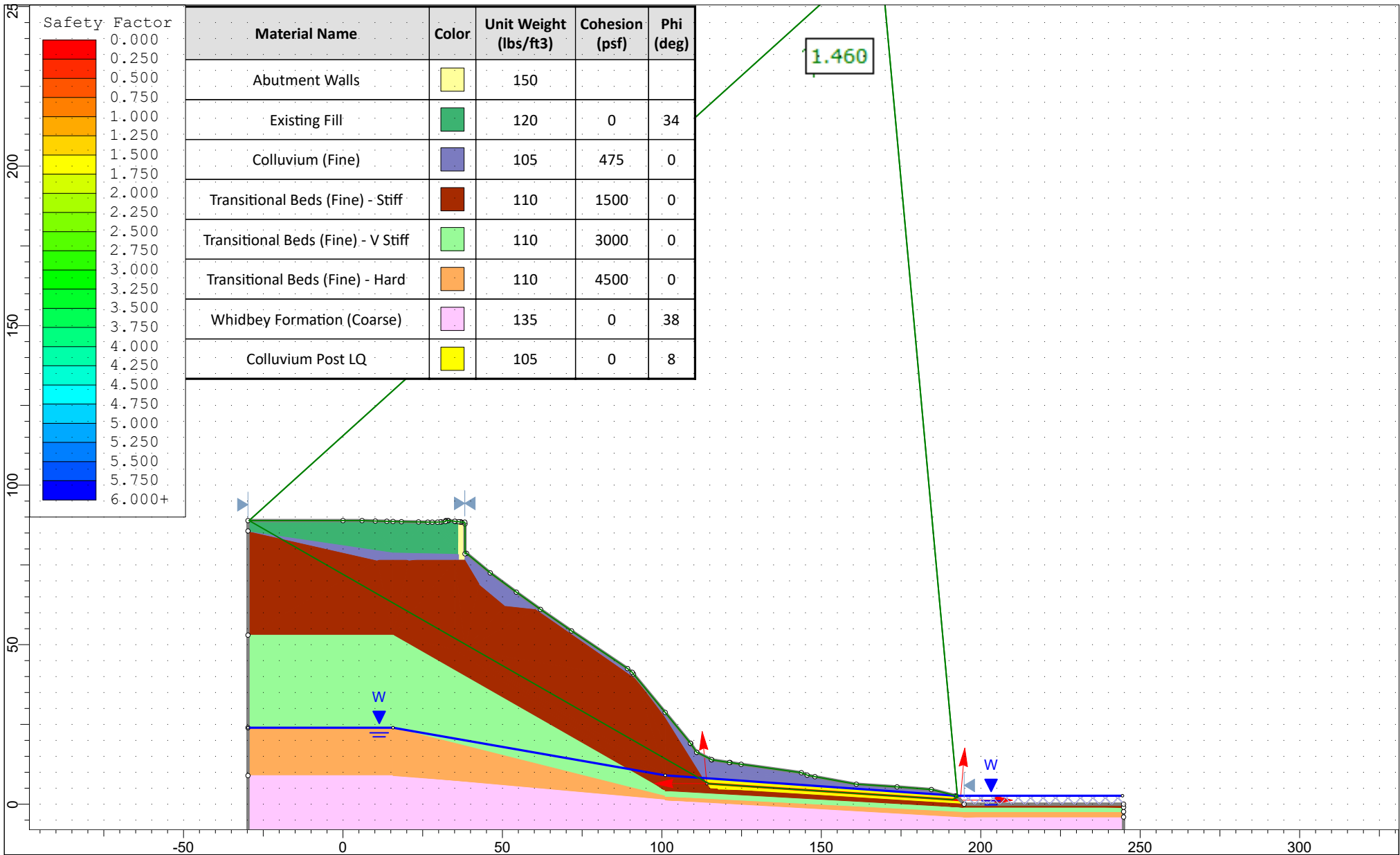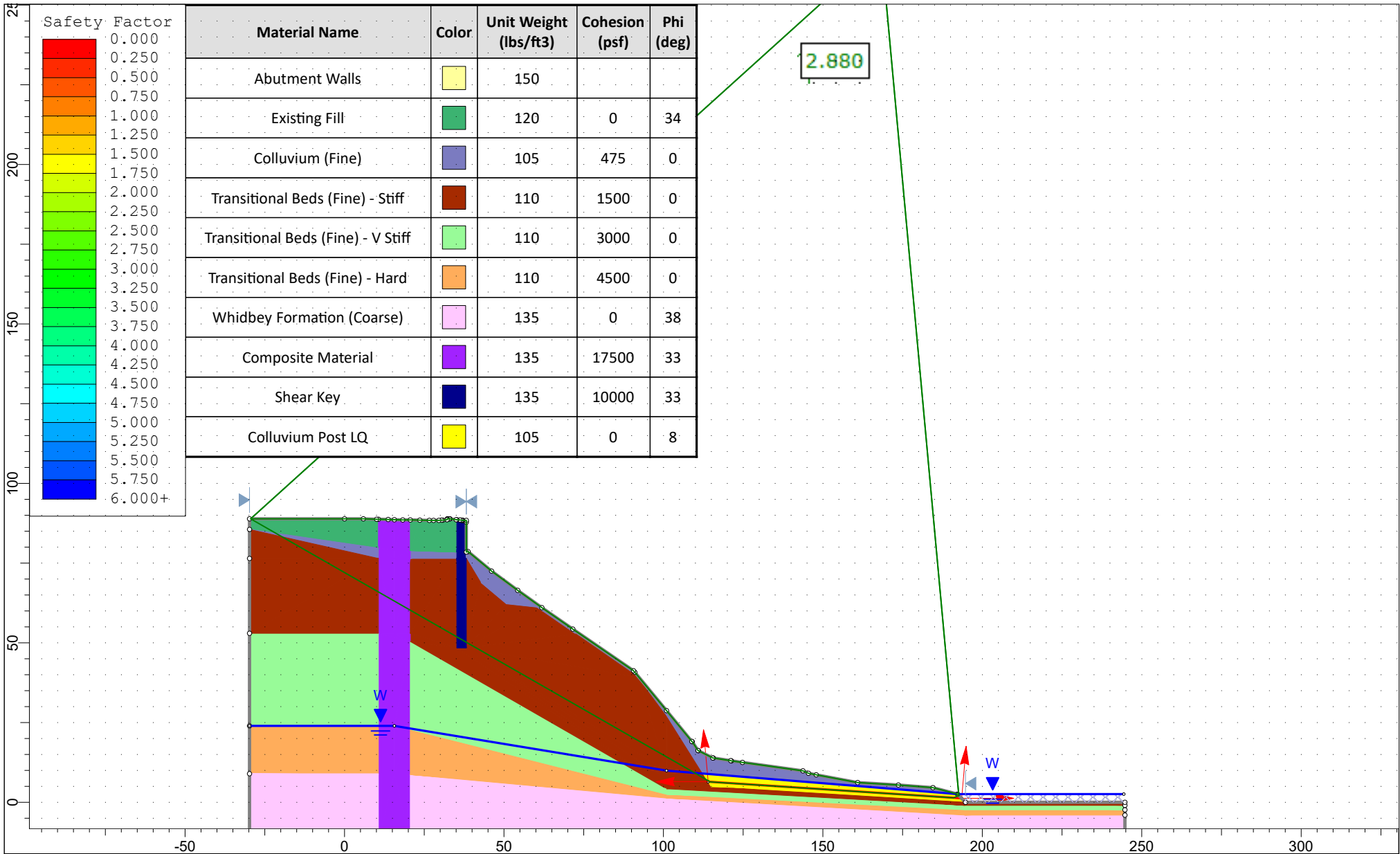|	
Project	Edgewater Creek Bridge Replacement
Analysis Description	Figure E-9 - Cross Section C-C' - Global Slope Stability Analysis - L to R
Drawn By	SKS
Scale	1:500
Company	HWA GeoSciences, Inc.
Date	10/9/2020
Loading Scenario	Pseudostatic Analysis - Proposed Conditions - Ravine Slope

HWA GeoSciences Inc.
DBE/MWBE

SLIDEINTERPRET 8.029

Safety Factor

0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+

1.460

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Abutment Walls		150		
Existing Fill		120	0	34
Colluvium (Fine)		105	475	0
Transitional Beds (Fine) - Stiff		110	1500	0
Transitional Beds (Fine) - V Stiff		110	3000	0
Transitional Beds (Fine) - Hard		110	4500	0
Whidbey Formation (Coarse)		135	0	38
Colluvium Post LQ		105	0	8

W

W

HWA | GEOSCIENCES INC.
DBE/MWBE

SLIDEINTERPRET 8.029

Project	Edgewater Creek Bridge Replacement			
Analysis Description	Figure E-10 - Cross Section C-C' - Global Slope Stability Analysis - L to R			
Drawn By	SKS	Scale	1:500	Company	HWA GeoSciences, Inc.
Date	10/9/2020	Loading Scenario	Post LQ Analysis - Existing Conditions	

Safety Factor

	0.000
	0.250
	0.500
	0.750
	1.000
	1.250
	1.500
	1.750
	2.000
	2.250
	2.500
	2.750
	3.000
	3.250
	3.500
	3.750
	4.000
	4.250
	4.500
	4.750
	5.000
	5.250
	5.500
	5.750
	6.000+

Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Abutment Walls		150		
Existing Fill		120	0	34
Colluvium (Fine)		105	475	0
Transitional Beds (Fine) - Stiff		110	1500	0
Transitional Beds (Fine) - V Stiff		110	3000	0
Transitional Beds (Fine) - Hard		110	4500	0
Whidbey Formation (Coarse)		135	0	38
Composite Material		135	17500	33
Shear Key		135	10000	33
Colluvium Post LQ		105	0	8

2.880

W

W

HWA GEOSCIENCES INC.
DBE/MWBE

SLIDEINTERPRET 8.029

Project	Edgewater Creek Bridge Replacement			
Analysis Description	Figure E-11 - Cross Section C-C' - Global Slope Stability Analysis - L to R			
Drawn By	SKS	Scale 1:500	Company	HWA GeoSciences, Inc.
Date	10/9/2020		Loading Scenario	Post LQ Analysis - Proposed Conditions

APPENDIX F

LPILE PARAMETER

B-1 (East Abutment) - LPILE Parameters

Existing Ground Surface Elevation at BH-1 = 125 Feet

Elevation at top of unit (ft)	Soil Layer	Soil Type (p-y model)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Effective Unit Wt, y' (pcf)[1]	Friction Angle (deg)	Undrained Shear Strength C_u (psf)[2]	p-y Modulus Static, k (pci)	p-y Modulus Seismic, k (pci)	Strain Factor, ε_{50} (dim)
125	Existing Fill	Sand (Reese)	0		120.0	34	--	90	90	--
				10	120.0	34	--	90	90	--
115	Transitional Beds (Above GWT)	Stiff Clay without Free Water (Reese)	10		110.0	--	1,500	500	200	0.007
				70	110.0	--	4,500	1,500	650	0.0045
55	Transitional Beds (Below GWT)	Stiff Clay with Free Water (Reese)	70		47.6	--	4,500	1,500	650	0.0045
				80	47.6	--	6,000	2,000	800	0.004
45	Whidbey Formation	Sand (Reese)	80		72.6	38	--	125	125	--
				101	72.6	38	--	125	125	--

1: Total Unit Weight (pcf) = Effective Unit Weight + 62.4 (for layers below water table)
2: Undrained Shear Strength, C = Cu = Su

B-4 (West Abutment) - LPILE Parameters

Existing Ground Surface Elevation at BH-4 = 120 Feet

Elevation at top of unit (ft)	Soil Layer	Soil Type (p-y model)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Effective Unit Wt, y' (pcf)[1]	Friction Angle (deg)	Undrained Shear Strength C_u (psf)[2]	p-y Modulus Static, k (pci)	p-y Modulus Seismic, k (pci)	Strain Factor, ε_{50} (dim)
120	Existing Fill	Sand (Reese)	0		120.0	34	--	90	90	--
				10	120.0	34	--	90	90	--
110	Colluvium	Soft Clay (Matlock)	10		105.0	--	475	70	--	0.015
				12.5	105.0	--	475	70	--	0.015
107.5	Transitional Beds (Above GWT)	Stiff Clay without Free Water (Reese)	12.5		110.0	--	1500	500	200	0.007
				70	110.0	--	4500	1500	650	0.0045
50	Transitional Beds (Below GWT)	Stiff Clay with Free Water (Reese)	70		47.6	--	4500	1500	650	0.0045
				80	47.6	--	6000	2000	800	0.004
40	Whidbey Formation	Sand (Reese)	80		72.6	38	--	125	125	--
				101.5	72.6	38	--	125	125	--

1: Total Unit Weight (pcf) = Effective Unit Weight + 62.4 (for layers below water table)
2: Undrained Shear Strength, C = Cu = Su

Figure F-2. Bridge Span Supports

B-2A (East Span Support) - LPILE Parameters

Existing Ground Surface Elevation at BH-2A = 64 Feet (60 feet below bridge deck)

Elevation at top of unit (ft)	Soil Layer	Soil Type (p-y model)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Effective Unit Wt, y' (pcf)[1]	Friction Angle (deg)	Undrained Shear Strength C_u (psf)[2]	p-y Modulus Static, k (pci)	p-y Modulus Seismic, k (pci)	Strain Factor, ε_{50} (dim)
64	Colluvium[3]	Soft Clay (Matlock)	0		105.0	--	475	70	--	0.0150
				10	105.0	--	475	70	--	0.0150
54	Transitional Beds[3]	Stiff Clay with Free Water (Reese)	10		47.6	--	1,500	500	200	0.007
				20	47.6	--	3,000	1,000	400	0.005
44	Transitional Beds	Stiff Clay with Free Water (Reese)	20		47.6	--	3,000	1,000	400	0.005
				33	47.6	--	4,500	1,500	650	0.0045
31	Whidbey Formation	Sand (Reese)	33		72.6	38	--	125	125	--
				76	72.6	38	--	125	125	--

1: Total Unit Weight (pcf) = Effective Unit Weight + 62.4 (for layers below water table)

2: Undrained Shear Strength, C = Cu = Su

3: We recommend that zero lateral resistance be assumed from the top of the proposed shafts until a depth at which the shaft face is a minimum of 4 shaft diameters away from the face of the adjacent slope (e.g. approximately 20 feet down for a 3-meter shaft).

B-3A (West Span Support) - LPILE Parameters

Existing Ground Surface Elevation at BH-3A = 62 Feet (60 feet below bridge deck)

Elevation at top of unit (ft)	Soil Layer	Soil Type (p-y model)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Effective Unit Wt, y' (pcf)[1]	Friction Angle (deg)	Undrained Shear Strength C_u (psf)[2]	p-y Modulus Static, k (pci)	p-y Modulus Seismic, k (pci)	Strain Factor, ε_{50} (dim)
62	Colluvium[3]	Soft Clay (Matlock)	0		105.0	--	475	70	--	0.0150
				12	105.0	--	475	70	--	0.0150
50	Transitional Beds[3]	Stiff Clay with Free Water (Reese)	12		47.6	--	1,500	500	200	0.007
				20	47.6	--	3,000	1,000	400	0.005
42	Transitional Beds	Stiff Clay with Free Water (Reese)	20		47.6	--	3,000	1,000	400	0.005
				25	47.6	--	4,500	1,500	650	0.0045
37	Whidbey Formation	Sand (Reese)	25		72.6	38	--	125	125	--
				75.25	72.6	38	--	125	125	--

1: Total Unit Weight (pcf) = Effective Unit Weight + 62.4 (for layers below water table)

2: Undrained Shear Strength, C = Cu = Su

3: We recommend that zero lateral resistance be assumed from the top of the proposed shafts until a depth at which the shaft face is a minimum of 4 shaft diameters away from the face of the adjacent slope (e.g. approximately 20 feet down for a 3-meter shaft).

4: Liquefiable soils are observed to be present extending from an elevation of approximately 49 to 51. However, due to the recommendation that zero lateral resistance be assumed extending to a depth of 20 feet below ground surface, the presence of this material can be neglected for the purpose of design.